Hydraulics Manual

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Washington State Department of Transportation

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Chapter 1 Design Policy

1-1 General

Various types of drainage facilities are required to protect the highway against surface and subsurface water. Drainage facilities must be designed to convey the water across, along, or away from the highway in the most economical, efficient and safe manner without damaging the highway or adjacent property. The purpose of this manual is to provide detailed information on the subjects of hydrologic and hydraulic analysis related to highway design. This manual should be used in conjunction with the Washington State Department of Transportation (WSDOT) Highway Runoff Manual and the WSDOT Design Manual, specifically Section 1210.

Developers, external agencies, utilities, etc., designing stormwater facilities within WSDOT right of way, shall assume the same responsibility as the Project Engineer's Office and prepare hydraulic reports as described in Section 1-3 of this manual. Additionally, pipes and stormwater treatment features (bioswale, pond, etc.) on WSDOT ROW are considered utility structures. Therefore, anytime such a feature is located on WSDOT ROW, a utility permit will be required. For more information on utility permits, designers should consult the *Utility Manual*, the *Agreements Manual* and or the *Developer Services Manual*.

The chapters contained in this manual, provide information necessary to complete hydrologic and hydraulic analysis for nearly all the situations that will be encountered during normal highway design. When a designer encounters a situation that is not described in this manual, the Regional Hydraulics Engineer or the Headquarters (HQ) Hydraulics Branch should be contacted for assistance. This manual is a summary of the FHWA's Hydraulic Engineering Circulars, for situations not discussed in this manual; designers can also consult FHWA's web site (http://www.fhwa.dot.gov/bridge/hydpub.htm). Designers are encouraged to request assistance as soon as questions or problems arise in a project, this will reduce the amount of redesign and if applicable allows more alternative solutions for the final design.

Designers should always keep in mind the legal and ethical obligations of WSDOT concerning hydraulic issues. The final project design should be carefully examined to determine if the project causes any significant changes to existing stormwater runoff and natural drainage facilities both upstream and downstream of the project. Care must be taken to ensure that the highway construction does not interfere with or damage any of these facilities.

1-2 Responsibility

The Project Engineer's Office is responsible for the preparation of correct and adequate drainage design. Actual design work may be performed in the Project Engineer's Office, by another WSDOT office, or by a private consulting engineer; however, in all cases, it is Project Engineer's responsibility to ensure that the design work is completed and that a hydraulic report is prepared as described in Section 1-3 of this manual. The hydraulic report should be completed during the early stages of design to allow adequate time for review prior to final Plans, Specifications and Estimates (PS&E) preparation. The Project Engineer's Office is also responsible for initiating the application for hydraulic related permits required by various local, state, and federal agencies.

While the Region is responsible for the preparation of hydraulic reports and PS&E for all drainage facilities except bridges, assistance from the HQ Hydraulics Branch may be requested for any drainage facility design, including the following:

1. Hydraulic design of unique drainage facilities (siphons, channel changes, etc.).
2. Structural design of hydraulic structures (culvert headwalls, fish ladders, etc.).
3. Analysis of streambank erosion and migration and design of stabilization countermeasures.
4. Special hydrological analysis (how snowmelt will/will not be considered, storm frequency prediction, etc.).
5. Analysis of closed drainage basins.
6. Providing the Washington State Attorney General's office with technical assistance on hydraulic issues.

The HQ Hydraulics Branch takes primary responsibility in the following areas:

1. Ensuring that the information in the WSDOT Hydraulics Manual is accurate and current.
2. Ensuring that the engineering related information in the WSDOT Highway Runoff Manual is accurate and current.
3. Hydraulic analysis of bridges, including hydraulic conveyance, floodplain impacts, deck drainage, and foundation scour.
4. Hydraulic and structural design of all large span corrugated metal culverts.
5. Hydraulic design of large span concrete culverts.
6. Hydraulic design of pumping facilities.
7. Flood plain studies and river hydraulic analysis.
8. Maintaining WSDOT Standard Plans involving drainage items.
9. Design of water supply and sewage disposal systems for safety rest areas. The Project Engineer's Office is responsible for contacting individual fire districts to collect local standards and forward the information onto HQ Hydraulics.
10. Reviewing and approving Type A Hydraulic Reports.
11. Providing the Regions with technical assistance on hydraulic issues that are the primary responsibility of the Region.
12. Providing basic hydrology and hydraulics training material to the Regions. Either Regional or HQ personnel will perform actual training. See the HQ Hydraulics web page for information on course availability: (http://www.wsdot.wa.gov/eesc/design/hydraulics/training.htm)

1-3 Hydraulic Reports

The hydraulic report is intended to serve as a complete documented record containing the engineering justification for all drainage modifications that occur as a result of the project. The primary use for a hydraulic report will be to allow review of the design and to assist in the preparation of the PS&E. The writer should approach the hydraulic report from the position of its defense in a court of law. It should be clearly written and show conditions before and after construction. The final copy of the hydraulic report must be stamped with a professional engineer's seal and signed by the project engineer.

1-3.1 Report Types

Hydraulic reports will be one of three types; Type A, Type B or a Hydraulic Summary. Figure 1-3 provides guidance for selecting the report type, however the Regional Hydraulics Engineer should be consulted to verify which type will be required.

<table>
<tr><th rowspan="2">Type of Report</th><th rowspan="2">Description</th><th colspan="2">Approval</th><th rowspan="2">PE Stamp</th></tr>
<tr><th>Regional</th><th>HQ</th></tr>
<tr><td>A</td><td>Projects with any of the following components:
• Culverts greater than 48 inches
• Over 10,000[1] sqft of new impervious area
• Detention facilities with a capacity equal to or greater than 1 acre-foot
• Storm sewer systems with greater than 10 hydraulic structures
• Channel realignment and or modifications (including fish passage)
• Bridge replacement projects
• Any fills in floodways
• Fills that include an excess of 1,000 cubic yards of displacement in the floodway fringe.
• Rest Area modifications and pump stations</td><td>X</td><td>X</td><td>X</td></tr>
<tr><td>B[2]</td><td>Projects with any of the following components:
• Culverts less than or equal to 48 inches in diameter
• Less than or equal to 10,000 square feet in new impervious area
• Detention facilities with less than 1 acre-foot in capacity
• Storm sewer systems with 10 or less hydraulic structures</td><td>X</td><td></td><td>X</td></tr>
</table>

1. Regions with Hydraulic Engineers who have served for 2 or more years, can petition the HQ Hydraulics Office to review some traditional type A reports without HQ approval. Projects must have less than 100,000 sqft of new impervious surface and include only methodologies that are at least 2 years old. Petitions should be submitted in writing, with both the Region Hydraulics Engineer and Regional Administrator signatures. This rule only applies to stormwater deviations, if projects have more than 10,000 and less than 100,000 sqft of new impervious surface plus other components listed for Type A reports, the report will still require HQ approval.
2. At the Regions discretion smaller projects may replace a Type B report with a Hydraulic Summary, see the Region Hydraulics Engineer for more information.

Hydraulic Report Selection Table
Figure 1-3

1-3.2 Submittal Package

Hard copies of these reports shall be sent to each of the approving authorities and one copy should be kept in the Project Engineer's Office for reference during plan preparation and construction. Each hard copy shall include a CD with the entire report contents, including software models. In the header or footer area of each document, please insert the file names and dates to be displayed on hard copies. The copy sent to the Regional Hydraulics Engineer will become WSDOT's permanent copy, and should be kept on file in the Region archives for an indefinite period of time. The copy sent to the HQ Hydraulics Branch will be kept for at least 10 years.

At times, a Hydraulics Report may need to be revised due to various elements within a proposed project. There are two ways to submit a change:

1. Revision – A revision is a correction of the existing report either due to an error or omitted design documentation. A revision can be approved under the original stamp. The designer should submit the revision along with a new title page, stamped and signed by the PE with the same date or later as the revision.
2. Supplement – A supplement is a change that was not part of the original scope of work. The same approval process is required as with the original report, however the supplement should be a stand-alone document that references the original report.

Either type of change should be prepared documenting the changes to be made with backup documentation. Include revised plans, calculations, and other updates as warranted in a submittal package to the Regional Hydraulics Engineer. An approval/concurrence letter will be issued for the supplement.

1-3.3 Report Contents

1. Since many different types of projects are designed and constructed by WSDOT, there is no set format for a hydraulic report. The hydraulic report must be well organized and contain all of the information necessary such that an engineer with no prior knowledge of the project could read the hydraulic report and fully understand the design. Items that should be included in a hydraulic report when applicable to the project are:Title page listing the project name, associated State Route, associated milepost(s), project number, and name, signature, and professional civil engineer license stamp of the project engineer.
2. Table of contents.
3. Vicinity map with the location of the project clearly shown.
4. An introduction, which discusses the hydraulic features of the project and why they are being installed. The project site conditions and layout should also be discussed. Photographs are often helpful in describing the site. There is no need to discuss the overall purpose of the project unless it is pertinent to some of the decisions made during design of the hydraulic features. The designer should state which stormwater management guidelines are used for quantity flow control, quality treatment, and enhanced quality treatment. Other design requirements such as local agency guidelines, WDFW fish passage, ESA, etc., should also be noted if applicable or differ from WSDOT design guidelines. An Environmental Documentation spread sheet is provided in Appendix 1-1-2 and should be included in the report appendices. The spreadsheet should reference why environmental decisions were made, who made them and any references used.

5. Discussion of site conditions as observed during inspection(s) of the site by the designer. This discussion will serve to confirm what is shown on maps and site plans, and it will note any features that will influence drainage design but are different than shown on maps and site plans.

6. Discussion of soil testing that has been performed at the site. This usually includes soil pH and resisitivity to determine acceptable pipe alternates, and soil infiltration and groundwater level for stormwater BMP design. Detailed descriptions of the following items: downstream analysis, wetland hydrology analysis, flow control and permanent BMP's. See Highway Runoff Manual, Chapter 2 minimum requirements for more information.

7. Hydrologic and hydraulic design calculations for all hydraulic features (e.g., culverts, storm drains, stormwater BMPs, inlets, gutters, ditches, streambank stabilization). Calculations should include: the actual numerical calculations, a discussion of what assumptions were made to perform the calculations and how the input parameters were determined. The calculations should always include enough supporting information to allow reviewers to completely duplicate the process used through the original design; however, excessive data which duplicates information already provided can often make the calculation process less understandable. Whenever possible calculation methodologies described in this manual should be followed. If a different method is selected, the reason for not using the standard WSDOT method should be explained and approved prior to submitting the report. Figures from this manual, standard WSDOT design forms, and suggested software should be used whenever possible to make the presentation of the information uniform throughout WSDOT. Actual calculations, design forms and output from software used in the project design should be included as part of the report appendices. Visit the following web link for a description of current programs and download information. http://www.wsdot.wa.gov/eesc/design/hydraulics/downloads.htm

8. A summary table of quantity flow control and quality treatment. The table should include pre-project and post-project pervious and impervious surface areas verses post-project pervious, new and replaced impervious surface areas. Additionally, the percentage of new and replaced impervious surface area and retrofitted existing impervious surface area addressed for quantity flow control, quality treatment, and enhanced quality treatment according to the stormwater management guidelines used for this report. It is recommended that the table be organized by alignment. Plans sheets should be included in the report appendix that highlight, number, and show measurements and areas of pervious and impervious basins and sub basins with flow direction arrows to each of their roadway drainage features. Include a structure note for each roadway drainage feature and use the same structure note in the hydrologic and hydraulic calculations.

9. Maps showing offsite drainage basins significant to the project should be included in the report appendices. The maps should show the entire drainage basins with flow direction arrows, including portions that are off WSDOT right of way. If the project has several drainage basins, which contribute to various hydraulic features, then each drainage basin should be clearly labeled and the same label should be referred to in the hydrologic and hydraulic calculations. When the change between existing and post construction conditions is important to the calculations, the maps should show both conditions, on separate maps if necessary for clarity. Maps should always be of an adequate scale to allow reviewers to verify all information used in the calculations.

10. The maximum junction spacing as approved by Region Maintenance and described in section 6-2.
11. A discussion on how or if snow was considered in the design if applicable. See Chapter 2 for further design guidance.
12. Any stormwater outfalls that leave WSDOT right of way must be entered into the Outfall Database and noted in the hydraulics report. (If no stormwater outfalls leave WSDOT right of way, the designer should also note that). Designers should include the following items in the database: State Route, milepost and offset, latitude/longitude, inventory date (mm/dd/yy), and the hydraulics report name. At this time the database has limited accessibility. Until the database is fully operational, the information detailed above should be sent directly to the HQ Water Quality Team Lead at tvetenr@wsdot.wa.gov or 360-570-6648. See the Environmental Affairs Office of Water Quality web site for more information: http://www.wsdot.wa.gov/environment/wqec/default.htm.
13. Traffic analysis data (only necessary for safety rest area designs).
14. Preliminary plans for the project showing locations of all the hydraulic features as well as the roadway cross sections should be included in the report appendices. These plans may change before final PS&E but should be of the same quality that is used for final PS&E. In some situations, combining drainage basin maps with these plans will improve overall clarity.
15. Profiles of all culverts, storm drains, stormwater BMPs, ditches, channels and the road should be included in the report appendices. Similar to the plans, these should be done to the same quality as final PS&E drawings.

If the designer is uncertain of how to organize a hydraulic report, the Regional Hydraulics Engineer should be consulted for information. For any type of project, the Regional Hydraulics Engineer should be able to provide an example of a hydraulic report from previous projects. The HQ Hydraulics Branch can also be contacted for information regarding the best way to prepare a hydraulic report.

1-4 Storm Frequency Policy

The design of a hydraulic structure requires an investigation to determine the runoff from the drainage area contributing flow. The amount of runoff from a drainage area will vary depending on the storm frequency that is being analyzed. The less frequent the storm is, the greater the associated precipitation will be and thus the greater the runoff will be.

Ideally every hydraulic structure would be designed for the largest possible amount of flow that could ever occur. Unfortunately this would require unusually large structures and would add an unjustifiable amount of cost to the projects; therefore hydraulic structures are analyzed for a specific storm frequency. When selecting a storm frequency for design purposes, consideration is given to the potential degree of damage to the roadway and adjacent property, potential hazard and inconvenience to the public, the number of users on the roadway, and the initial construction cost of the hydraulic structure.

The way in which these factors interrelate can become quite complex. WSDOT policy regarding design storm frequency for typical hydraulic structures has been established so the designer does not have to perform a risk analysis for each structure on each project. The design storm frequency is referred to in terms of mean recurrence interval (MRI) of precipitation or high flows.

MRI is the average interval between events equal to or greater than a given event. It can also be viewed as the probability that such an event will occur in any one year. For example, a peak flow having a 25-year recurrence interval has a 4 percent probability of being equaled or exceeded in any future year. A peak flow having a 2-year recurrence interval has a 50 percent probability of being equaled or exceeded in any future year. The greater the MRI, the lower the probability that the event will occur in any given year.

It is important to keep in mind that MRI does not indicate that events occur on a time schedule. MRI cannot be used to predict time of occurrence. Each event is independent of all others, so the chance that a 25-year peak flow will occur this year remains the same regardless of what flows occurred last year. The correct way to view MRI is that it predicts the average occurrence of events over an extended period of time. For example, a 25-year peak discharge is expected to be equaled or exceeded 4 times in 100 years.

Figure 1-4 lists the recommended MRIs for design of hydraulic structures. Based on past experience, these will give acceptable results in most cases. Occasionally the cost of damages may be so great, or the level of services using the roadway may be so important, that a higher MRI is appropriate. Good engineering judgment must be used to recognize these instances and the design modified accordingly. In high-risk areas a statistical risk analysis (benefit/cost) may be needed to arrive at the most suitable frequency.

Type Of Structure	MRI (Years)
Gutters	10
Storm Drain Inlets - On Longitudinal Slope	10
Storm Drain Inlets - Vertical Curve Sag	50
Storm Drain Laterals	25
Storm Drain Trunk Lines	25
Ditches	10
Standard Culverts - Design For HW/D Ratio	25
Standard Culverts - Check For High Flow Damage	100
Bottomless Culverts - Design For HW Depth	25 & 100
Bridges - Design For Flow Passage And Foundation Scour	100
Bridges - Check For High Flow Damage	500

Design Frequency for Hydraulic Structures
Figure 1-4

1-5 Schedule

WSDOT has developed the Project Delivery Information System (PDIS) to track and manage projects. PDIS utilizes a Master Deliverables List (MDL) to identify major elements that occur during most projects. The MDL is intended to be a starting point for creating a work breakdown structure (WBS) and identifies specific offices the designer should communicate with during the development of the project schedule. Figure 1-5 summarizes estimated time requirements for hydraulic review, however times can vary depending on; staffing, project impacts and complexity of the project. Additionally this should not be a substitution for communication, before finalizing the project schedule, designers should verify time lines with the offices mentioned below.

Report Type	Region Review	HQ Review	Description
A	4 weeks	8 weeks	Assumes Regional review and approval precedes HQ review. The 8-week HQ review includes 4 weeks for initial review, 2 weeks for designer changes and 2 weeks for final review.
B	4 weeks	N/A	Assumes 2 weeks for review and if needed 1 week for changes and 1 week for final review.
	2 weeks	4 weeks	Both HQ and Regional Hydraulics should be contacted as soon as it is determined a Supplemental Report is required.
Design Build	-	-	Coordinate with Field Operations Support Service Center (FOSSC), HQ Hydraulics and Region Hydraulics.

Estimated Review Duration for Project Schedules
Figure 1-4

The design team should determine preparation time for a project. Both the Region and HQ Hydraulic Offices can provide assistance if needed.

More information on PDIS can be found at the following web site:
(🖰 http://wwwi.wsdot.wa.gov/projects/PDIS/)

English to Metric Conversions	English to English Conversions	Metric to Metric Conversions
	Length	
1 inch = 25.4 millimeters 1 foot = 0.3048 meters 1 mile = 1.609 kilometers 1 yard = 0.914 meters	1 mile = 5,280 feet 1 yard = 3 feet	1 centimeter = 10 millimeters 1 meter = 100 centimeters 1 kilometer = 100 meters
	Area	
1 square inches = 645.16 sq. millimeters 1 square feet = 0.093 sq. meters 1 acres = 0.4047 hectares 1 square miles = 2.59 square kilometers	1 acre (acre ft) = 43,560 sq. feet 1 sq. mile = 640 acres 1 sq. mile = 1 section of land	1 sq. centimeter = 100 sq. millimeters 1 sq. meter = 10000 sq. centimeters 1 hectare = 10,000 sq. meters 1 square kilometer = 1000000 sq. meters
	Volume	
1 ounce = 29.57 milliliters 1 gallon = 3.785 liters 1 cubic foot = 0.0283 cubic meters 1 acre-foot = 1,233.6 cubic meters	1 cubic foot= 7.48 gallons 1 acre-foot = 43,560 cubic feet	1 cubic centimeter = 1000 cubic millimeters 1 cubic meter = 1000000 cubic centimeters 1 cubic meter = 1000 liters
	Flowing Water Rates:	
1 cubic foot/second = 0.0283 cubic meters/second 1 cubic foot/second = 28.32 liters/second	1 cubic foot/second = 448.83 gallons/minute 1 cubic foot/second = 0.646 million gal./day 1 cubic foot/second = 1.984 acre-feet per day	
	Pressure	
1 pound force = 4.45 Newtons 1 pound force/sq.in = 6.89 kilopascals 1 foot of water = 2.988 Kilopascals 1 atmosphere = 101.4 Kilopascals	1 foot of water = 0.433 pounds/square in. 1 foot of water = 62.4 pounds/square ft. 1 atmosphere = 14.70 pounds/square in. 1 atmosphere = 33.94 feet of water	
	Mass	
1 ounces = 28.35 grams 1 pounds = 0.454 kilograms	1 ton = 2000 pounds	1 kilogram = 1000 grams 1 tonne = 1000 kilograms
	Temperature	
°F = 1.8*°C + 32	N/A	N/A

Appendix 1-2

ENVIRONMENTAL DOCUMENTATION

PROJECT NUMBER AND TITLE:

ITEM #	DATE	LOCATION (MP)	DESCRIPTION	RESOLUTION	REFERENCES	OWNER	APPROVED BY

2-1 General Hydrology

The Washington State Department of Transportation (WSDOT) Headquarters (HQ) Hydraulics Branch uses several methods of determining runoff rates and/or volumes. Experience has shown them to be accurate, convenient, and economical. The following methods will be discussed in detail in subsequent sections of this chapter:

1. The Rational Method
2. The Santa Barbara Urban Hydrograph (SBUH) Method
3. Published Flow Records
4. United States Geological Survey (USGS) Regression Equations
5. Flood Reports

   Two other methods, documented testimony and high water mark observations, may be used as back-up material to confirm the results of the above statistical and empirical methods. Where calculated results vary from on-site observations, further investigation may be required. The additional two methods are:

6. Documented Testimony

   Documented testimony of long-time residents should also be given serious consideration by the designer. The engineer must be aware of any bias that testifying residents may have. Independent calculations should be made to verify this type of testimony. The information that may be furnished by local residents of the area should include, but not be limited to the following:

   a. Dates of past floods.
   b. High water marks.
   c. Amount of drift.
   d. Any changes in the river channel which may be occurring (i.e., stability of streambed, is channel widening or meandering?).
   e. Estimated velocity.
   f. Description of flooding characteristics between normal flow to flood stage.

7. High Water Mark Observations

   Sometimes the past flood stage from a drainage area may be determined by observing ordinary high water marks (OHWM) on existing structures or on the bank of a stream or ditch. The Region Biologist can assist in determining the OHWM if needed. These marks along with other data may be used to determine the discharge by methods discussed in the Open Channel Flow chapter or the Culverts chapter of this manual.

Additional hydrologic procedures are available including complex computer models which can give the designer accurate flood predictions. However, these methods, which require costly field data and large amounts of data preparation and calculation time, can rarely be justified for a single hydraulic structure. The HQ Hydraulics Branch should be contacted before a procedure not listed above is used in a hydrologic analysis.

For the sake of simplicity and uniformity, the HQ Hydraulics Branch will normally require the use of one of the first five of the seven methods listed above. Exceptions will be permitted if adequate justification is provided.

2-2 Selecting a Method

Each of the first five methods listed above are appropriate to use for different design conditions and none of the methods will cover all situations. The first step in performing a hydrologic analysis is to determine which method is most appropriate. Generally there is no need to select more than one method.

1. Rational Method: This method is used when peak discharges for small basins must be determined. It is a fairly simple and accurate method especially when the basin is primarily impervious. The rational method is appropriate for culvert design, pavement drainage design, storm drain design, and some stormwater facility designs.
2. SBUH Method: This method is used when peak discharges and runoff volumes for small basins must be determined. This method is not complicated but requires a computer due to its computationally intensive nature. The SBUH method is required for many stormwater facility designs and can also be used for culvert design, pavement drainage design, and storm drain design.
3. Published Flow Records: This method is used when peak discharges for large basins must be determined. This is more of a collection of data rather than a predictive analysis like the other methods listed. Some agencies (primarily the USGS) gather streamflow data on a regular basis. This collected data can be used to predict flood flows for the river and is typically more accurate than calculated flows. Published flow records are most appropriate for culvert and bridge design.
4. USGS Regression Equations: This method is used when peak discharges for medium to large basins must be determined. It is a set of regression equations that were developed using data from streamflow gaging stations. The regression equations are very simple to use but lack the accuracy of published flow records. USGS regression equations are appropriate for culvert and bridge design.
5. Flood Reports: This method is used when peak discharges for medium to large basins must be determined. It is basically using results from an analysis that has been conducted by another agency. Often these values are very accurate since they were developed from an in-depth analysis. Flood report data are appropriate for culvert and bridge design.

2-3 Drainage Basin

The size of the drainage basin is one of the most important parameters regardless of which method of hydrologic analysis is used. To determine the basin area, select the best available topographic map or maps which cover the entire area contributing surface runoff to the point of interest. Outline the area on the map or maps and determine the size in square meters, acres, or square miles (as appropriate for the specific equations), either by scaling or by using a planimeter. Sometimes drainage basins are small enough that they fit entirely on the CADD drawings for the project. In these cases the basin can be digitized on the CADD drawing and calculated by the computer. Any areas within the basin that are known to be non-contributing to surface runoff should be subtracted from the total drainage area.

The USGS has published two open-file reports titled, Drainage Area Data for Western Washington and Drainage Area Data for Eastern Washington. Copies of these reports can be obtained from the HQ Hydraulics Branch and the Regional Hydraulics Engineer. These reports list drainage areas for all streams in Washington where discharge measurements have been made. Drainage areas are also given for many other sites such as highway crossings, major stream confluences, and at the mouths of significant streams. These publications list a total of over 5,000 drainage areas and are a valuable time saver to the designer. The sites listed in these publications are usually medium sized and larger drainage basin areas. Small local drainage areas need to be determined from topographic maps as outlined above.

2-4 Cold Climate Considerations

Snowmelt and rain-on-snow is a complicated process, which can result in greater rates of runoff than occur from rainfall alone. Winter rainfall hydrographs added to snowmelt need to be compared to spring and summer rainfalls, which are likely to occur without the snow pack. The HQ Hydraulics office is developing guidance on how to handle these issues, which will be available in the September 2004 revision of the *Hydraulics Manual*. In the interim, designers should consider the following issues and consult HQ or Regional Hydraulics for further design guidance.

1. Roadside Drainage - During the design phase, consideration should be given to how roadside snow will accumulate and possibly block inlets and other flow paths for water present during the thawing cycle. If it is determined that inlets could be blocked by the accumulation of plowed snow, consideration should be given to an alternate courses of travel for runoff.
2. Retention Ponds - When retention ponds are located near the roadway, the emergency spillway should be located outside of any snow storage areas that could block overflow passage, or an alternative flow route should be designated.
3. Frozen Ground - Frozen Ground coupled with snowmelt or rain on snow can cause unusually adverse conditions. These combined sources of runoff are generally reflected in the USGS regression equations as well as in the historic gauge records. No corrections or adjustments typically need to be made to these hydrology methods for frozen ground or snowmelt. For smaller basins, the SBUH and Rational methods are typically used to determine peak volume and peak runoff rates. The CN value for the SBUH method, and the runoff coefficient for the Rational method can be increased to account for frozen ground. See section 2.5.2 for further guidance.

2-5 The Rational Method

2-5.1 General

The rational method is used to predict peak flows for small drainage areas which can be either natural or developed. The rational method can be used for culvert design, pavement drainage design, storm drain design, and some stormwater facility design. The greatest accuracy is obtained for areas smaller than 40 hectares (100 acres) and for developed conditions with large areas of impervious surface (e.g., pavement, roof tops, etc.). Basins up to 400 hectares (1,000 acres) may be evaluated using the rational formula; however, results for large basins often do not properly account for effects of infiltration and thus are less accurate. Designers should never perform a rational method analysis on a basin that is larger than the lower limit specified for the USGS regression equations since the USGS regression equations will yield a more accurate flow prediction for that size of basin.

The formula for the rational method is:

$$Q = \frac{CIA}{K_c}$$

where: Q = Runoff in cubic meters per second (cubic feet per second for English units)

C = Runoff coefficient in dimensionless units

I = Rainfall intensity in millimeters per hour (inches per hour)

A = Drainage area in hectares (acres)

K_c = Units conversion factor of 360 (1 for English units)

When several subareas within a drainage basin have different runoff coefficients, the rational formula can be modified as follows:

$$Q = \frac{I(\Sigma CA)}{K_c}$$

Where: ΣCA = $C_1 \times A_1 + C_2 \times A2 + \ldots C_n \times A_n$

Hydrologic information calculated by the rational method should be submitted on DOT Form 235-009 (see Figure 2-4.1). This format contains all the required input information as well as the resulting discharge. The description of each area should be identified by name or stationing so that the reviewer may easily locate each area.

2-5.2 Runoff Coefficients

The runoff coefficient "C" represents the percentage of rainfall that becomes runoff. The rational method implies that this ratio is fixed for a given drainage basin. In reality, the coefficient may vary with respect to prior wetting and seasonal conditions. The use of an average coefficient for various surface types is quite common and it is assumed to stay constant through the duration of the rainstorm.

Frozen ground can cause a dramatic increase in the runoff coefficient. When this condition is coupled with heavy rainfall and, perhaps, melting snow, the runoff can be much greater than calculated values that did not account for these conditions. This condition is common for larger basins that are above 300 m (1000 ft.) in elevation and is automatically accounted for in the USGS regression equations. For small basins where the rational method is being used, the designer should increase the runoff coefficient to reflect the reduction in infiltration and resulting increased surface runoff.

In a high growth rate area, runoff factors should be projected that will be characteristic of developed conditions 20 years after construction of the project. Even though local storm water practices (where they exist) may reduce potential increases in runoff, prudent engineering should still make allowances for predictable growth patterns.

The coefficients in Figure 2-4.2 are applicable for peak storms of 10-year frequency. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. Generally, when designing for a 25-year frequency, the coefficient should be increased by 10 percent; when designing for a 50-year frequency, the coefficient should be increased by

20 percent; and when designing for a 100-year frequency, the coefficient should be increased by 25 percent. The runoff coefficient should not be increased above 0.95, unless approved by the Regional Hydraulics Engineer. Higher values may be appropriate for steeply sloped areas and/or longer return periods, because in these cases infiltration and other losses have a proportionally smaller effect on runoff.

SR	Project	
Calculated By		Date

EQUATIONS	LEGEND	
$T_c = \frac{L}{K\sqrt{S}} = \frac{L^{1.5}}{K\sqrt{\Delta H}}$ $I = \frac{m}{(T_c)^n}$ $Q = \frac{CIA}{K_c}$	Q = Flow	T_c = Time of concentration
	L = Length of drainage basin	m & n = Rainfall coefficients
	S = Average slope	K_c = Conversion
	K = Ground cover coefficient	C = Runoff coefficient
	ΔH = Change in elevation of basin	A = Drainage area

Description of Area	MRI	L	H	S	K	T_c	Rainfall Coeff		K_c	C	I	A	Q
							m	n					

Hydrology by the Rational Method
Below is the web link for electronic spreadsheet
http://www.wsdot.wa.gov/eesc/design/hydraulics/programs/hydrology.xls
Figure 2-4.1

Type of Cover	**Flat**	**Rolling 2%-10%**	**Hilly Over 10%**
Pavement and Roofs	0.90	0.90	0.90
Earth Shoulders	0.50	0.50	0.50
Drives and Walks	0.75	0.80	0.85
Gravel Pavement	0.50	0.55	0.60
City Business Areas	0.80	0.85	0.85
Suburban Residential	0.25	0.35	0.40
Single Family Residential	0.30	0.40	0.50
Multi Units, Detached	0.40	0.50	0.60
Multi Units, Attached	0.60	0.65	0.70
Lawns, Very Sandy Soil	0.05	0.07	0.10
Lawns, Sandy Soil	0.10	0.15	0.20
Lawns, Heavy Soil	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay and Loam	0.50	0.55	0.60
Cultivated Land, Sand and Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	0.90
Parks and Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodland and Forests	0.10	0.15	0.20
Meadows and Pasture Land	0.25	0.30	0.35
Pasture with Frozen Ground	0.40	0.45	0.50
Unimproved Areas	0.10	0.20	0.30

Runoff Coefficients for the Rational Method — 10-Year Return Frequency

Figure 2-4.2

2-5.3 Time of Concentration

If rainfall is applied at a constant rate over a drainage basin, it would eventually produce a constant peak rate of runoff. The amount of time that passes from the moment that the constant rainfall begins to the moment that the constant rate of runoff begins is called the time of concentration. This is the time required for the surface runoff to flow from the most hydraulically remote part of the drainage basin to the location of concern.

Actual precipitation does not fall at a constant rate. A precipitation event will begin with small rainfall intensity then, sometimes very quickly, build to peak intensity and eventually taper down to no rainfall. Because rainfall intensity is variable, the time of concentration is included in the rational method so that the designer can determine the proper rainfall intensity to apply across the basin. The intensity that should be used for design purposes is the highest intensity that will occur with the entire basin contributing flow to the location where the designer is interested in knowing the flow rate. It is important to note that this may be a much lower intensity than the absolute maximum intensity. The reason is that it often takes several minutes before the entire basin is contributing flow but the absolute maximum intensity lasts for a much shorter time so the rainfall intensity that creates the greatest runoff is less than the maximum by the time the entire basin is contributing flow.

Most drainage basins will consist of different types of ground covers and conveyance systems that flow must pass over or through. These are referred to as flow segments. It is common for a basin to have flow segments that are overland flow and flow segments that are open channel flow. Urban drainage basins often have flow segments that are flow through a storm drain pipe in addition to the other two types. A travel time (the amount of time required for flow to move through a flow segment) must be computed for each flow segment. The time of concentration is equal to the sum of all the flow segment travel times.

For a few drainage areas, a unique situation occurs where the time of concentration that produces the largest amount of runoff is less than the time of concentration for the entire basin. This can occur when two or more subbasins have dramatically different types of cover (i.e., different runoff coefficients). The most common case would be a large paved area together with a long narrow strip of natural area. In this case, the designer should check the runoff produced by the paved area alone to determine if this scenario would cause a greater peak runoff rate than the peak runoff rate produced when both land segments are contributing flow. The scenario that produces the greatest runoff should be used, even if the entire basin is not contributing flow to this runoff.

The procedure for determining the time of concentration for overland flow was developed by the United States Natural Resources Conservation Service (formerly known as the Soil Conservation Service) and is described below. It is sensitive to slope, type of ground cover, and the size of channel. If the total time of concentration is less than 5 minutes, a minimum of five minutes should be used as the duration, see section 2-4.4 for details. The time of concentration can be calculated as follows:

$$T_t = \frac{L}{K\sqrt{S}} = \frac{L^{1.5}}{K\sqrt{\Delta H}}$$

$$T_c = T_{t1} + T_{t2} + \ldots + T_{tnz}$$

where:

T_t = Travel time of flow segment in minutes

T_c = Time of concentration in minutes

L = Length of segment in meters (feet for English units)

ΔH = Elevation change across segment in meters (feet)

K = Ground cover coefficient in meters per minute (feet per minute)

S = Slope of segment $\frac{\Delta H}{L}$ in meters per meter (feet per feet)

Type of Cover		K (metric)	K (English)
Forest with heavy ground cover		50	150
Minimum tillage cultivation		75	280
Short pasture grass or lawn		125	420
Nearly bare ground		200	600
Small roadside ditch w/grass		275	900
Paved area		375	1,200
Gutter flow	100 mm deep	450	1,500
	150 mm deep	725	2,400
	200 mm deep	950	3,100
Storm Sewers	300 mm diam	925	3,000
	450 mm diam.	1,200	3,900
	600 mm diam.	1,425	4,700
Open Channel Flow (n = .040)	300 mm deep	350	1,100
Narrow Channel (w/d =1)	600 mm deep	550	1,800
	1.20 m deep	850	2,800
Open Channel Flow (n =.040)	300 mm deep	600	2,000
Wide Channel (w/d =9)	600 mm deep	950	3,100
	1.20 m deep	1,525	5,000

Ground Cover Coefficients
Figure 2-4.3

2-5.4 Rainfall Intensity

After the appropriate storm frequency for the design has been determined (see Chapter 1) and the time of concentration has been calculated, the rainfall intensity can be calculated. Designers should never use a time of concentration that is less than 5 minutes for intensity calculations, even when the calculated time of concentration is less than 5 minutes. The 5-minute limit is based on two ideas:

1. Shorter times give unrealistic intensities. Many IDF curves are constructed from curve smoothing equations and not based on actual data collected at intervals shorter than 15 to 30 minutes. To make the curves shorter, involves extrapolation, which is not reliable.
2. It takes time for rainfall to generate into runoff within a defined basin, thus it would not be realistic to have less than 5 minutes for a time of concentration.

It should be noted that the rainfall intensity at any given time is the average of the most intense period enveloped by the time of concentration and is not the instantaneous rainfall. The equation for calculating rainfall intensity is:

$$I = \frac{m}{(T_c)^n}$$

where: I = Rainfall intensity in millimeters per hour (inches per hour in English units)

T_c = Time of concentration in minutes

I = Rainfall intensity in millimeters per hour (inches per hour)

m and n = Coefficients in dimensionless units (see Figures 2-4.4A and 2-4.4B)

The coefficients (m and n) have been determined for all major cities for the 2-, 5-, 10-, 25-, 50-, and 100year mean recurrence intervals (MRI). The coefficients listed are accurate from 5-minute duration to 1,440-minute duration (24 hours). These equations were developed from the 1973 National Oceanic and Atmospheric Administration Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume IX-Washington.

The designer should interpolate between the two or three nearest cities listed in the tables when working on a project that is in a location not listed on the table. If the designer must do an analysis with a T_c greater than 1,440 minutes, the rational method should not be used.

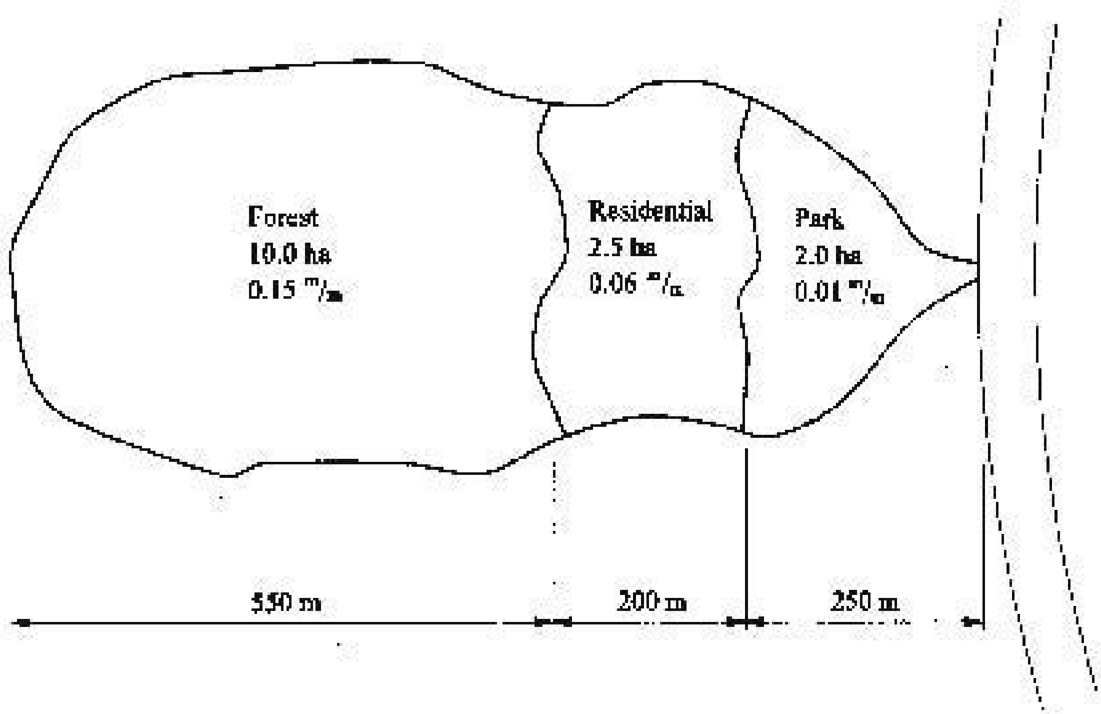
	2-Year MRI		5-Year MRI		10-Year MRI		25-Year MRI		50-Year MRI		00-Year MRI	
Location	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**
Aberdeen and Hoquiam	129	0.488	158	0.488	179	0.487	208	0.487	229	0.487	250	0.487
Bellingham	109	0.549	142	0.555	167	0.559	201	0.562	226	0.563	251	0.565
Bremerton	96	0.480	123	0.487	143	0.490	170	0.494	190	0.496	210	0.498
Centralia and Chehalis	92	0.506	123	0.518	146	0.524	178	0.530	201	0.533	225	0.537
Clarkston and Colfax	128	0.628	174	0.633	209	0.635	256	0.638	291	0.639	325	0.639
Colville	83	0.558	138	0.593	177	0.610	230	0.626	271	0.635	311	0.642
Ellensburg	73	0.590	132	0.631	179	0.649	240	0.664	287	0.672	335	0.678
Everett	94	0.556	132	0.570	160	0.575	199	0.582	228	0.585	256	0.586
Forks	106	0.410	130	0.412	148	0.413	172	0.414	190	0.415	208	0.416
Hoffstadt Cr. (SR 504)	101	0.448	132	0.462	156	0.469	189	0.476	214	0.480	238	0.484
Hoodsport	114	0.428	138	0.428	157	0.427	182	0.428	200	0.428	219	0.428
Kelso and Longview	108	0.507	140	0.515	164	0.519	197	0.524	221	0.526	246	0.529
Leavenworth	77	0.530	105	0.542	143	0.575	202	0.594	248	0.606	281	0.611
Metaline Falls	85	0.527	124	0.553	155	0.566	189	0.570	236	0.592	265	0.591
Moses Lake	66	0.583	128	0.634	178	0.655	243	0.671	295	0.681	346	0.688
Mt. Vernon	100	0.542	133	0.552	159	0.557	193	0.561	218	0.564	245	0.567
Naselle	116	0.432	144	0.441	156	0.432	190	0.443	204	0.440	226	0.436
Olympia	97	0.466	123	0.472	143	0.474	168	0.477	188	0.478	208	0.480
Omak	77	0.583	129	0.618	168	0.633	222	0.647	263	0.654	304	0.660
Pasco and Kennewick	73	0.590	132	0.631	178	0.649	240	0.664	287	0.672	335	0.678
Port Angeles	109	0.530	138	0.531	159	0.531	187	0.532	208	0.532	229	0.532
Poulsbo	97	0.506	126	0.513	149	0.516	178	0.519	200	0.521	222	0.523
Queets	108	0.422	132	0.423	149	0.423	172	0.423	190	0.423	208	0.424
Seattle	90	0.515	123	0.531	143	0.530	175	0.539	200	0.545	222	0.545
Sequim	89	0.551	127	0.569	156	0.577	195	0.585	226	0.590	255	0.593
Snoqualmie Pass	92	0.417	122	0.435	167	0.459	196	0.459	223	0.461	259	0.476
Spokane	88	0.556	138	0.591	177	0.609	231	0.626	271	0.635	313	643
Stevens Pass	120	0.462	155	0.470	208	0.500	217	0.484	269	0.499	316	513
Tacoma	91	0.516	121	0.527	145	0.533	176	0.539	200	0.542	223	545
Vancouver	74	0.477	103	0.496	125	0.506	154	0.515	177	0.520	199	0.525
Walla Walla	85	0.569	141	0.609	185	0.627	246	0.645	291	0.653	337	0.660
Wenatchee	80	0.535	124	0.566	157	0.579	202	0.592	237	0.600	271	0.605
Yakima	98	0.608	149	0.633	187	0.644	239	0.654	278	0.659	317	0.663

Index to Rainfall Coefficients (Metric Units)
Figure *2-4.4 A*

	2-Year MRI		5-Year MRI		10-Year MRI		25-Year MRI		50-Year MRI		100-Year MRI	
Location	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**	**m**	**n**
Aberdeen and Hoquiam	5.10	0.488	6.22	0.488	7.06	0.487	8.17	0.487	9.02	0.487	9.86	0.487
Bellingham	4.29	0.549	5.59	0.555	6.59	0.559	7.90	0.562	8.89	0.563	9.88	0.565
Bremerton	3.79	0.480	4.84	0.487	5.63	0.490	6.68	0.494	7.47	0.496	8.26	0.498
Centralia and Chehalis	3.63	0.506	4.85	0.518	5.76	0.524	7.00	0.530	7.92	0.533	8.86	0.537
Clarkston and Colfax	5.02	0.628	6.84	0.633	8.24	0.635	10.07	0.638	11.45	0.639	12.81	0.639
Colville	3.48	0.558	5.44	0.593	6.98	0.610	9.07	0.626	10.65	0.635	12.26	0.642
Ellensburg	2.89	0.590	5.18	0.631	7.00	0.649	9.43	0.664	11.30	0.672	13.18	0.678
Everett	3.69	0.556	5.20	0.570	6.31	0.575	7.83	0.582	8.96	0.585	10.07	0.586
Forks	4.19	0.410	5.12	0.412	5.84	0.413	6.76	0.414	7.47	0.415	8.18	0.416
Hoffstadt Cr. (SR 504)	3.96	0.448	5.21	0.462	6.16	0.469	7.44	0.476	8.41	0.480	9.38	0.484
Hoodsport	4.47	0.428	5.44	0.428	6.17	0.427	7.15	0.428	7.88	0.428	8.62	0.428
Kelso and Longview	4.25	0.507	5.50	0.515	6.45	0.509	7.74	0.524	8.70	0.526	9.67	0.529
Leavenworth	3.04	0.530	4.12	0.542	5.62	0.575	7.94	0.594	9.75	0.606	11.08	0.611
Metaline Falls	3.36	0.527	4.90	0.553	6.09	0.566	7.45	0.570	9.29	0.592	10.45	0.591
Moses Lake	2.61	0.583	5.05	0.634	6.99	0.655	9.58	0.671	11.61	0.681	13.63	0.688
Mt. Vernon	3.92	0.542	5.25	0.552	6.26	0.557	7.59	0.561	8.60	0.564	9.63	0.567
Naselle	4.57	0.432	5.67	0.441	6.14	0.432	7.47	0.443	8.05	0.440	8.91	0.436
Olympia	3.82	0.466	4.86	0.472	5.62	0.474	6.63	0.477	7.40	0.478	8.17	0.480
Omak	3.04	0.583	5.06	0.618	6.63	0.633	8.74	0.647	10.35	0.654	11.97	0.660
Pasco and Kennewick	2.89	0.590	5.18	0.631	7.00	0.649	9.43	0.664	11.30	0.672	13.18	0.678
Port Angeles	4.31	0.530	5.42	0.531	6.25	0.531	7.37	0.532	8.19	0.532	9.03	0.532
Poulsbo	3.83	0.506	4.98	0.513	5.85	0.516	7.00	0.519	7.86	0.521	8.74	0.523
Queets	4.26	0.422	5.18	0.423	5.87	0.423	6.79	0.423	7.48	0.423	8.18	0.424
Seattle	3.56	0.515	4.83	0.531	5.62	0.530	6.89	0.539	7.88	0.545	8.75	0.5454
Sequim	3.50	0.551	5.01	0.569	6.16	0.577	7.69	0.585	8.88	0.590	10.04	0.593
Snoqualmie Pass	3.61	0.417	4.81	0.435	6.56	0.459	7.72	0.459	8.78	0.461	10.21	0.476
Spokane	3.47	0.556	5.43	0.591	6.98	0.609	9.09	0.626	10.68	0.635	12.33	0.643
Stevens Pass	4.73	0.462	6.09	0.470	8.19	0.500	8.53	0.484	10.61	0.499	12.45	0.513
Tacoma	3.57	0.516	4.78	0.527	5.70	0.533	6.93	0.539	7.86	0.542	8.79	0.545
Vancouver	2.92	0.477	4.05	0.496	4.92	0.506	6.06	0.515	6.95	0.520	7.82	0.525
Walla Walla	3.33	0.569	5.54	0.609	7.30	0.627	9.67	0.645	11.45	0.653	13.28	0.660
Wenatchee	3.15	0.535	4.88	0.566	6.19	0.579	7.94	0.592	9.32	0.600	10.68	0.605
Yakima	3.86	0.608	5.86	0.633	7.37	0.644	9.40	0.654	10.93	0.659	12.47	0.663

Index to Rainfall Coefficients (English Units)

Figure 2-4.4B



2-5.5 Rational Formula Example

Compute the 25-year runoff for the Olympia watershed shown above. Three types of flow conditions exist from the highest point in the watershed to the outlet. The upper portion is 10.0 hectares of forest cover with an average slope of 0.15 m/m. The middle portion is 2.5 hectares of single family residential with a slope of 0.06 m/m and primarily lawns. The lower portion is a 2.0 hectares park with 450 mm storm sewers with a general slope of 0.01 m/m.

$$T_c = \sum \frac{L}{K\sqrt{S}} = \frac{550}{50\sqrt{0.15}} + \frac{200}{125\sqrt{0.06}} + \frac{250}{1{,}200\sqrt{0.01}}$$

$$T_c = 28\,\text{min} + 7\,\text{min} + 2\,\text{min} = 37\,\text{min}$$

$$I = \frac{m}{(T_c)^n} = \frac{168}{(37)^{0.477}} = 30.0\frac{mm}{hr}$$

$$\sum CA = 0.20(10.0ha) + 0.40(2.5ha) + 0.10(2.0ha) = 3.2ha$$

$$Q = \frac{I\left(\sum CA\right)}{K_c} = \frac{(30.0)(3.2)}{360} = 0.27\frac{m^3}{s}(9.4cfs)$$

2-6 Santa Barbara Urban Hydrograph Method

When designing flood control structures and some stormwater treatment facilities, the designer must know more than just the peak flow that will occur. Along with the peak flow, the volume of runoff must be calculated as well as the relationship between time and the rate of runoff. The only way to accomplish this is to use a method of analysis that incorporates a hydrograph. A hydrograph is a graphical representation of flow versus time.

Of the several commonly accepted hydrograph methods, the Santa Barbara Urban Hydrograph (SBUH) method is the best suited for the types of projects that WSDOT designs. It was developed to calculate flows from small to medium sized urban basins using input data that is readily available and equations that are easily understood. While not all WSDOT projects are in urban basins, it is typically the paved surfaces (similar to urban areas) that generate the majority of the total flow.

The SBUH method is computationally intensive. Calculations for even a single drainage area would take hours if done by hand. Because of this, the only practical way to perform an analysis is to use a computer application. The equations used are simple enough to be incorporated into a spreadsheet, which would provide accurate calculations; however, it is highly recommended that one of the commercially available computer programs that include the SBUH method be used. The advantage of using commercial software is the overall consistency of input and output formats and the reliability obtained from being tested in several different design circumstances.

There are several commercially available computer programs that include the SBUH method. Each of these programs has certain features that make them unique from other programs but the primary calculations are performed the same way. Because of this, nearly any commercially available computer program that includes the SBUH method is acceptable for designing WSDOT projects.

The Washington State Ferries Division and each WSDOT Region have purchased site licenses for the computer program Storm Shed so this program and associated manual are available to all WSDOT designers. The HQ Hydraulics Branch encourages the use of Storm Shed whenever performing an SBUH method analysis and is available to lend technical assistance.

The SBUH method only calculates flow that will occur from surface runoff and thus is not accurate for large drainage basins where ground water flow can be a major contributor to the total flow. As a result, the SBUH method is most accurate for drainage basins smaller than 40 hectares (100 acres) and should never be used for drainage basins larger than 400 hectares (1,000 acres).

Chapter 4 of the WSDOT *Highway Runoff Manual* discusses the details of performing an analysis using the SBUH method. The *Highway Runoff Manual* also includes information on flood control structures and stormwater treatment facilities that will be used in conjunction with almost all SBUH method analyses.

2-7 Published Flow Records

When available, published flow records provide the most accurate data for designing culverts and bridge openings. This is because the values are based on actual measured flows and not calculated flows. The streamflows are measured at a gaging site for several years. A statistical analysis (typically Log Pearson Type III) is then performed on the measured flows to predict the recurrence intervals.

The USGS maintains a large majority of the gaging sites throughout Washington State. A list of all of the USGS gages that have adequate data to develop the recurrence intervals and their corresponding flows is provided in Appendix 2-1. In addition to these values, the HQ Hydraulics Branch maintains records of daily flows and peak flows for all of the current USGS gages. Also, average daily flow values for all current and discontinued USGS gages are available through the Internet on the USGS homepage (note that these are average daily values and not peak values). http://www.nwis.waterdata.usgs.gov/wa/nwis/dvstat

Some local agencies also maintain streamflow gages. Typically, these are on smaller streams than the USGS gages. While the data obtained from these gages is usually of high enough quality to use for design purposes, the data is not always readily available. If the designer thinks that there is a possibility that a local agency has flow records for a particular stream then the engineering department of the local agency should be contacted. The HQ Hydraulics Branch does not maintain a list of active local agency streamflow gages.

2-8 USGS Regression Equations

While measured flows provide the best data for design purposes, it is not practical to gage all rivers and streams in the state. A set of equations has been developed by the USGS to calculate flows for drainage basins that do not have a streamflow gage. The equations were developed by performing a regression analysis on streamflow gage records to determine which drainage basin parameters are most influential in determining peak runoff rates.

The equations break the state up into nine unique hydrologic regions. A map of the regions can be found in Appendix 2-2. The various hydrologic regions require different input variables so the designer should determine which set of equations will be used before gathering data for the analysis. Appendix 2-2 also contains precipitation information that is required input for many of the equations. Other input parameters such as total area of the drainage basin, percent of the drainage basin that is in forest cover, and percent of the drainage basin that is in lakes, swamps, or ponds will need to be determined by the designer through use of site maps, aerial photographs, and site inspections.

The equations are listed in Figures 2-7.1 through 2-7.9. Each figure contains one set of equations for a hydrologic region of the state. Each figure also lists the statistical accuracy of the individual equations and describes the required input parameters for the equations and their limits of usage. The designer should be careful not to use data that is outside of the limits specified for the equations since the accuracy of the equations is unknown beyond these points.

The designer must be aware of the limitations of these equations. They were developed for natural basins so any drainage basin that has been urbanized should not be analyzed with this method. Also any river that has a dam and reservoir in it should not be analyzed with these equations. Finally, the designer must keep in mind that due to the simple nature of these equations and the broad range of each hydrologic region, the results of the equations contain a fairly wide confidence interval, represented as the standard error.

The standard error is a statistical representation of the accuracy of the equations. Each equation is based on many rivers and the final result represents the mean of all the flow values for the given set of basin characteristics. The standard error shows how far out one standard deviation is for the flow that was just calculated. For a bell- shaped curve in statistical analysis, 68 percent of all the samples are contained within the limits set by one standard deviation above the mean value and one standard deviation below the mean value. It can also be viewed as indicating that 50 percent of all the samples are equal to or less than the flow calculated with the equation and 84 percent of all samples are equal to or less than one standard deviation above the flow just calculated.

The designers shall use the mean value determined from the regression equations with no standard error or confidence interval. If the flows are too low or too high for that basin based on information that the designer has collected, then the designer may apply the standard error specific to the regression equation accordingly. The designer should consult the Regional Hydraulic Engineer for assistance.

The equations were developed with data ranging through the 1992 water year. They represent updates to the USGS regression equations developed for Washington State in 1973 and the designer should disregard the previous version of the equations.

The equations are only presented in English units. To obtain metric flow data, the designer should input the necessary English units data into the appropriate regression equation and then multiply the results by 0.02832 to convert the final answer to cubic meters per second.

The HQ Hydraulics Branch has a computer program available for distribution that does the calculations for these equations.

2-9 Flood Reports

Flood reports have been developed for many rivers in Washington State. Most of these reports, and the ones that are most readily accessible, have been developed by the Federal Emergency Management Agency (FEMA). Other reports have been developed by the United States Army Corps of Engineers and by some local agencies.

These reports are a good source of flow information since they were developed to analyze the flows during flooding conditions of a particular river or stream. The types of calculations used by the agency conducting the analysis are more complex than the rational method or USGS regression equations and because of this are more accurate. The increased time required to perform these complex calculations is not justified for the typical structure that WSDOT is designing; however, if the analysis has already been performed by another agency, then it is in WSDOTs best interest to use this information. Flood study data should never be used in place of published flow records.

The HQ Hydraulics Branch maintains a complete set of FEMA reports and also has several Corps of Engineers flood reports. Regional Environmental Offices should be contacted for local agency reports.

2-10 Mean Annual Runoff

Sometimes it is necessary to determine the mean annual flow or runoff for a given stream. When published flow records are available they are the best source of information. Minor streams, which do not have any gaging records available, can be estimated by the following procedure:

Metric Units:

$$Q = \frac{(MAR)A}{1{,}241}$$

where: Q = Mean annual runoff in m^3/s

MAR = Mean annual runoff in inches taken from Appendix 2-2

A = Area of the drainage basin in square kilometers

English Units:

$$Q = \frac{(MAR)A}{13.6}$$

where: Q = Mean annual runoff in cfs

MAR = Mean annual runoff in inches taken from Appendix 2-2

A = Area of the drainage basin in square miles

Washington State Hydrology
USGS Regression Equations
Region 1 – 61 stations

SR ____________________ Date _____________

Project ___________________________________

Made By _________________________________

Equations:

Q 2yr	=	$0.35 \times A^{0.923} \times (MAP)^{1.24}$	(Standard Error = 32%)
Q 10yr	=	$0.50 \times A^{0.921} \times (MAP)^{1.26}$	(Standard Error = 33%)
Q 25yr	=	$0.59 \times A^{0.921} \times (MAP)^{1.26}$	(Standard Error = 34%)
Q 50yr	=	$0.666 \times A^{0.921} \times (MAP)^{1.26}$	(Standard Error = 36%)
Q 100yr	=	$0.745 \times A^{0.922} \times (MAP)^{1.26}$	(Standard Error = 37%)

Legend Limits

Q	=	Flow (cfs)
A	=	Drainage Basin Area (miles ^{2}) (0.15 miles $\leq s^2$ A$\leq$ 1,294 miles$^{2})$
MAP	=	Mean Annual Precipitation (inches) (45.0 in MAP $\leq$ 201 in)

Description of Area	Return Frequency	A	P2	Q

USGS Regression Equations — Region 1
Figure 2-7.1

Washington State Hydrology
USGS Regression Equations
Region 2 – 202 stations

SR ____________________ Date _____________

Project ___________________________________

Made By _________________________________

Equations:

Q 2yr	=	$0.9 \times A^{0.877} \times (MAP)^{1.51}$	(Standard Error = 56)
Q 10yr	=	$0.129 \times A^{0.868} \times (MAP)^{1.57}$	(Standard Error = 53)
Q 25yr	=	$0.148 \times A^{0.864} \times (MAP)^{1.59}$	(Standard Error = 53)
Q 50yr	=	$0.161 \times A^{0.862} \times (MAP)^{1.61}$	(Standard Error = 53)
Q 100yr	=	$0.174 \times A^{0.861} \times (MAP)^{1.62}$	(Standard Error = 54)

Legend Limits

Q	=	Flow (cfs)	
A	=	Drainage Basin Area (miles ^{2}) (0.08 miles $\leq s^2$ A$\leq$ 3,020 miles$^{2)}$	
MAP	=	Mean Annual Precipitation (inches)	(23 in MAP $\leq$ 170 in)

Description of Area	Return	A	MAP	Q

USGS Regression Equations — Region 2
Figure 2-7.2

Washington State Hydrology
USGS Regression Equations
Region 3 – 63 stations

SR ____________________ Date _____________

Project ___________________________________

Made By _________________________________

Equations:

Q 2yr	=	$0.817 \times A^{0.877} \times (MAP)^{1.02}$	(Standard Error = 57%)
Q 10yr	=	$0.845 \times A^{0.875} \times (MAP)^{1.14}$	(Standard Error = 55%)
Q 25yr	=	$0.912 \times A^{0.874} \times (MAP)^{1.17}$	(Standard Error = 54%)
Q 50yr	=	$0.808 \times A^{0.872} \times (MAP)^{1.23}$	(Standard Error = 54%)
Q 100yr	=	$0.801 \times A^{0.871} \times (MAP)^{1.26}$	(Standard Error = 55%)

Legend		Limits	
Q	=	Flow (cfs)	
A	=	Drainage Basin Area (miles ^{2}) (0.36 miles $\leq s^2$ A$\leq$ 2,198 miles^2)	
MAP	=	Mean Annual Precipitation (inches)	(42 in MAP $\leq$ 132 in)

Description of Area	Return	A	MAP	Q

USGS Regression Equations — Region 3
Figure 2-7.3

Washington State Hydrology
USGS Regression Equations
Region 4 – 60 stations

SR ____________________ Date _____________

Project ___________________________________

Made By _________________________________

Equations:

Q 2yr	=	$0.025 \times A^{0.880} \times (MAP)^{1.70}$	(Standard Error = 82%)
Q 10yr	=	$0.179 \times A^{0.856} \times (MAP)^{1.37}$	(Standard Error = 84%)
Q 25yr	=	$0.341 \times A^{0.85} \times (MAP)^{1.26}$	(Standard Error = 87%)
Q 50yr	=	$0.505 \times A^{0.845} \times \times (MAP)^{1.20}$	(Standard Error = 90%)
Q 100yr	=	$0.703 \times A^{0.842} \times (MAP)^{1.15}$	(Standard Error = 92%)

Legend Limits

Q	=	Flow (cfs)
A	=	Drainage Basin Area (miles ^{2}) (0.66 miles $\leq s^{2}$ A$\leq$ 2,220 miles^{2})
MAP	=	Mean Annual Precipitation (inches) (12 in MAP $\leq$ 108 in)

Description of Area	Return	A	MAP	Q

USGS Regression Equations — Region 4
Figure 2-7.4

Washington State Hydrology
USGS Regression Equations
Region 5 – 19 stations

SR ____________________ Date _____________

Project ___________________________________

Made By _________________________________

Equations:

Q 2yr	=	$14.7 \times A^{0.815}$	(Standard Error = 96%)
Q 10yr	=	$35.2 \times A^{0.787}$	(Standard Error = 63%)
Q 25yr	=	$48.2 \times A^{0.779}$	(Standard Error = 56%)
Q 50yr	=	$59.1 \times A^{0.774}$	(Standard Error = 53%)
Q 100yr	=	$71.2 \times A^{0.769}$	(Standard Error = 52%)

Legend Limits

Q = Flow (cfs)

A = Drainage Basin Area (miles ^{2}) (0.38 miles $\leq s^2$ A$\leq$ 638 miles^2)

Description of Area	Return	A	S	Q

USGS Regression Equations — Region 5
Figure 2-7.5

Washington State Hydrology
USGS Regression Equations
Region 6 – 23 stations

SR ____________________ Date _____________

Project ____________________________________

Made By __________________________________

Equations:

Q 2yr	=	$2.24 \times A^{0.719} \times (MAP)^{0.833}$	(Standard Error = 63%)
Q 10yr	=	$17.8 \times A^{0.716} \times (MAP)^{0.487}$	(Standard Error = 69%)
Q 25yr	=	$38.6 \times A^{0.714} \times (MAP)^{0.359}$	(Standard Error = 72%)
Q 50yr	=	$63.6 \times A^{0.713} \times (MAP)^{0.276}$	(Standard Error = 74%)
Q 100yr	=	$100 \times A^{0.713} \times (MAP)^{0.201}$	(Standard Error = 77%)

Legend		Limits	
Q	=	Flow (cfs)	
A	=	Drainage Basin Area (miles ^{2}) (0.50 miles $\leq s^2$ A$\leq$ 1,297 miles$^{2)}$	
MAP	=	Mean Annual Precipitation (inches)	(10 in $\leq$ MAP $\leq$ 116 in)

Description of Area	Return	A	MAP	Q

USGS Regression Equations — Region 6
Figure 2-7.6

Washington State Hydrology
USGS Regression Equations
Region 7 – 17 stations

SR ____________________ Date _____________

Project ___________________________________

Made By _________________________________

Equations:

Q 2yr = $8.77 \times A^{0.629}$ (Standard Error = 128)

Q 10yr = $50.9 \times A^{0.587}$ (Standard Error = 63)

Q 25yr = $91.6 \times A^{0.574}$ (Standard Error = 54)

Q 50yr = $131 \times A^{0.566}$ (Standard Error = 53)

Q 100yr = $179 \times A^{0.558}$ (Standard Error = 56)

Legend Limits

Q = Flow (cfs)

A = Drainage Basin Area (miles ^{2})
(0.21 miles $\leq$s^2 A$\leq$ 2,228 miles^2)

Description of Area	Return Frequency	A	Q

USGS Regression Equations — Region 7
Figure 2-7.7

Washington State Hydrology
USGS Regression Equations
Region 8 – 23 stations

SR ____________________ Date _____________

Project ____________________________________

Made By ____________________________________

Equations:

$Q_{2yr} = 12.0 \times A^{0.761}$ (Standard Error = 133%)

$Q_{10yr} = 32.6 \times A^{0.706}$ (Standard Error = 111%)

$Q_{25yr} = 46.2 \times A^{0.687}$ (Standard Error = 114%)

$Q_{50yr} = 57.3 \times A^{0.676}$ (Standard Error = 119%)

$Q_{100yr} = 69.4 \times A^{0.666}$ (Standard Error = 126%)

Legend Limits

Q = Flow (cfs)

A = Drainage Basin Area (miles ^{2}) (0.59 miles $\leq s^2$ A$\leq$ 689 miles$^{2)}$

Description of Area	Return	A	F	Q

USGS Regression Equations — Region 8
Figure 2-7.8

Washington State Hydrology
USGS Regression Equations
Region 9 – 36 stations

SR ____________________ Date _____________

Project ___________________________________

Made By _________________________________

Equations:

Q 2yr	=	$0.803 \times A^{0.672} \times (MAP)^{1.16}$	(Standard Error = 80%)
Q 10yr	=	$15.4.0 \times A^{0.597} \times (MAP)^{0.662}$	(Standard Error = 57%)
Q 25yr	=	$41.1 \times A^{0.570} \times (MAP)^{0.508}$	(Standard Error = 55%)
Q 50yr	=	$74.7 \times A^{0.553} \times (MAP)^{0.420}$	(Standard Error = 55%)
Q 100yr	=	$126 \times A^{0.538} \times (MAP)^{.344}$	(Standard Error = 56%)

Legend		Limits	
Q	=	Flow (cfs)	
A	=	Drainage Basin Area (miles ^{2})	(0.54 miles $\leq s^2$ A$\leq$ 2500 miles$^{2)}$
MAP	=	Mean Annual Precipitation (inches)	(12.0 in MAP $\underline{\leq 40.0)}$

Description of Area	Return	A	P2	Q

USGS Regression Equations — Region 9
Figure 2-7.9

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12009500	Bear Branch Near Naselle, Wash	1964-79	1,130	2,000	2,490	2,880	3,300
12010000	Naselle River Near Naselle, Wash	1930-92	5,740	8,650	10,000	11,000	12,000
12010500	Salmon Creek Near Naselle, Wash	1954-65	1,730	2,320	2,640	2,870	3,110
12010600	Lane Creek Near Naselle, Wash	1950-70	176	219	236	248	258
12010700	South Fork Naselle River Near Naselle,Wash	1965-79	2,280	3,670	4,400	4,960	5,520
12010800	South Nemah River Near Naselle, Wash	1963-77	235	331	373	401	428
12011000	North Nemah River Near South Bend,Wash	1947-61 1965-68	1,400	1,770	1,930	2,030	2,140
12011100	North Nemah R Trib Near South Bend, Wash	1949-66	49	75	88	97	106
12011200	Williams Creek Near South Bend, Wash	1965-79	691	1,400	1,840	2,210	2,600
12011500	Willapa River at Lebam, Wash	1949-74	2,860	4,140	4,740	5,160	5,570
12012000	Fork Creek Near Lebam, Wash	1954-79	2,310	3,480	4,080	4,530	5,000
12012200	Green Creek Near Lebam, Wash	1950-69	126	196	234	264	295
12013500	Willapa River Near Willapa, Wash	1949-92	8,200	11,100	12,400	13,200	14,000
12014500	S Fk Willapa R Near Raymond, Wash	1954-79	1,640	2,760	3,350	3,800	4,260
12015100	Clearwater Creek Near Raymond, Wash	1965-79	265	495	623	724	829
12015500	North River Near Brooklyn, Wash	1954-65	1,660	2,560	2,980	3,280	3,570
12016700	Joe Creek Near Cosmopolis, Wash	1949-70	152	246	295	331	368
12017000	North River Near Raymond, Wash	1928-79	8,160	13,900	17,600	20,600	24,100
12019600	Water Mill Creek Near Pe Ell, Wash	1950-70	89	138	161	177	193
12020000	Chehalis River Near Doty, Wash	1940-92	9,570	16,800	21,200	24,700	28,500
12020500	Elk Creek Near Doty,Wash	1945-79	1,710	2,920	3,590	4,120	4,680
12020900	S Fork Chehalis River Near Boistfort, Wash	1966-80	2,700	4,660	5,760	6,630	7,540
12021000	South Fork Chehalis R at Boistfort, Wash	1945-65	3,200	4,550	5,200	5,690	6,160
12024000	S Fork Newaukum River Near Onalaska, Wash	1945-48 1958-79	2,280	3,480	4,030	4,420	4,790
12025000	Newaukum River NR Chehalis, Wash	1929-31 1943-92	5,840	8,900	10,300	11,300	12,300

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12025300	Salzer Creek Near Centralia, Wash	1969-79	303	325	332	337	341
12025700	Skookumchuck River Near Vail, Wash	1968-92	2,670	4,940	6,210	7,220	8,260
12026000	Skookumchuck River Near Centralia, Wash	1930-33 1940-69	3,590	5,390	6,230	6,840	7,420
12026150	Skookumchuck River Below Bldy Rn Creek	1971-92	2,880	6,460	8,480	10,000	11,600
12026300	Skookumchuck R Trib Near Bucoda, Wash	1960-75	35	57	68	76	84
12026400	Skookumchuck River Near Bucoda, Wash	1971-92	4,040	8,220	10,500	12,200	13,900
12027500	Chehalis River Near Grand Mound, Wash	1929-92	25,100	41,400	50,700	58,000	65,700
12029700	Chehalis River Near Oakville, Wash	1947-76	28,800	41,900	48,500	53,400	58,500
12030000	Rock Creek at Cedarville, Wash	1945-74	1,120	1,480	1,620	1,720	1,820
12031000	Chehalis River at Porter, Wash	1947-85 1987-92	29,200	42,400	49,000	54,000	59,000
12032500	Cloquallum River at Elma, Wash	1945-79	3,130	4,650	5,340	5,820	6,280
12034200	East Fork Satsop River Near Elma, Wash	1958-73	3,000	4,510	5,210	5,710	6,090
12034700	W Fork Satsop R Trib Near Matlock, Wash	1958-77	40	79	99	115	132
12035000	Satsop River Near Satsop, Wash	1930-92	24,300	37,200	43,100	47,300	51,300
12035400	Wynoochee River Near Grisdale, Wash	1973-92	6,080	9,950	11,700	13,000	14,200
12035450	Big Creek Near Grisdale, Wash	1973-92	2,240	3,280	3,780	4,140	4,490
12036000	Wynoochee River Near Aberdeen, Wash	1973-92	8,240	12,600	14,500	15,900	17,100
12036650	Anderson Creek Near Montesano, Wash	1973-85	254	436	532	605	680
12037400	Wynoochee River Near Montesano, Wash	1973-92	13,400	19,500	22,200	24,000	25,700
12038750	Gibson Creek Near Quinalt, Wash	1965-75	272	406	476	531	587
12039000	Humptulips River Near Humptulips, Wash	1934-35 1943-79	18,900	28,400	32,800	35,900	38,900
12039050	Big Creek Near Hoquiam, Wash	1949-70	60	108	134	154	175
12039100	Big Creek Tributary Near Hoquiam, Wash	1949-68	16	24	27	29	31
12039300	North Fork Quinalt R Near Amanda Pk, Wash	1965-86	16,100	27,200	32,800	37,000	41,000
12039400	Higley Creek Near Amanda Park, Wash	1955-74	174	300	366	417	468

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12039500	Quinalt River at Quinalt Lake, Wash	1912-22 1926-92	21,800	37,100	44,900	50,800	56,700
12040000	Clearwater River Near Clearwater, Wash	1938-67	19,800	30,700	36,300	40,600	45,000
12040500	Queets River Near Forks, Wash	1931-67 1975-92	64,600	97,400	114,000	126,000	138,000
12041000	Hoh River Near Forks, Wash	1927-64	18,500	29,600	35,300	39,600	44,000
12041200	Hoh River at US Hwy 101 Near Forks, Wash	1961-92	31,800	51,700	61,000	67,600	73,900
12041500	Sol Duc River Near Fairholm, Wash	1918-21 1934-80	9,460	16,200	19,800	22,500	25,200
12041600	Sol Duc River Tributary Near Fairholm, Wash	1956-75	26	48	60	69	80
12042700	May Creek Near Forks, Wash	1950-68	485	708	804	870	932
12042900	Grader Creek Near Forks, Wash	1950-88	323	481	555	607	658
12043000	Calawah River Near Forks, Wash	1976-80 1985-92	17,800	29,800	35,800	40,300	44,800
12043100	Dickey River Near La Push, Wash	1963-80	8,220	13,200	15,900	18,000	20,300
12043163	Sooes R Blw Miller Cr Near Ozette, Wash	1976-86	3,490	5,780	6,900	7,720	8,520
12043300	Hoko River Near Sekiu, Wash	1963-78	6,780	11,000	13,400	15,200	17,100
12043430	East Twin River Near Pysht, Wash	1963-78	940	1,280	1,420	1,510	1,600
12044000	Lyre River at Piedmont, Wash	1918-27	714	1,220	1,460	1,640	1,820
12045500	Elwha River Near Port Angeles, Wash	1927-92	13,300	24,200	29,900	34,100	38,400
12046800	East Valley Creek at Port Angeles, Wash	1950-63	21	45	59	70	81
12047100	Lees Creek at Port Angeles, Wash	1949-70	92	238	340	430	533
12047300	Morse Creek Near Port Angeles, Wash	1967-79	1,280	2,920	3,970	4,860	5,840
12047500	Siebert Creek Near Port Angeles, Wash	1953-69	430	1,330	1,990	2,580	3,250
12047700	Gold Creek Near Blyn,Wash	1965-75	65	133	172	203	235
12048000	Dungeness River Near Sequim, Wash	1924-30 1938-92	2,890	5,720	7,180	8,260	9,340
12049400	Dean Creek at Blyn, Wash	1949-70	27	60	80	96	114
12050500	Snow Creek Near Maynard, Wash	1953-79	209	480	640	768	901
12052400	Penny Creek Near Quilcene, Wash	1949-68	207	461	600	702	806

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12053000	Dosewallips River Near Brinnon, Wash	1931-68	4,350	8,180	10,300	11,900	13,600
12053400	Dosewallips R Tributary Near Brinnon, Wash	1951-70	38	57	67	74	81
12054000	Duckabush River Near Brinnon, Wash	1939-92	4,450	7,010	8,150	8,940	9,690
12054500	Hamma Hamma River Near Eldon, Wash	1952-79	3,460	5,560	6,560	7,280	7,990
12054600	Jefferson Creek Near Eldon, Wash	1958-79	2,300	3,110	3,420	3,610	3,790
12056300	Annas Bay Tributary Near Potlatch, Wash	1950-70	51	134	192	244	302
12056500	N Fk Skokomish R Near Hoodsport, Wash	1925-92	6,480	12,800	16,600	19,600	22,800
12057500	North Fork Skokomish R Near Hoodsport, Wash	1926-30	7,820	12,400	14,800	16,600	18,500
12058000	Deer Meadow Creek Near Hoodsport, Wash	1953-79	114	249	326	359	448
12059500	N Fk Skokomish River Near Potlatch, Wash	1945-92	1,970	4,430	6,000	7,320	8,770
12059800	S Fk Skokomish River Near Hoodsport, Wash	1964-79	3,700	5,570	6,360	6,910	7,410
12060000	S Fk Skokomish River Near Potlatch, Wash	1924-32 1947-64	8,520	15,400	19,100	21,900	24,800
12060500	South Fork Skokomish River Near Union, Wash	1932-84	11,800	18,300	21,100	23,100	25,000
12061200	Fir Creek Tributary Near Potlatch, Wash	1955-74	139	228	276	313	350
12061500	Skokomish River Near Potlatch, Wash	1944-92	16,100	23,400	26,900	29,400	31,800
12065500	Gold Creek Near Bremerton, Wash	1946-79	118	209	252	283	313
12066000	Tahuya River Near Bremerton, Wash	1946-56	331	510	591	648	702
12067500	Tahuya River Near Belfair, Wash	1946-56	709	1,030	1,190	1,300	1,410
12068500	Dewatto River Near Dewatto, Wash	1948-79	1,030	1,680	2,010	2,250	2,500
12069550	Big Beef Creek Near Seabeck, Wash	1970-81	551	767	861	927	990
12070000	Dogfish Creek Near Poulsbo, Wash	1948-73	129	219	270	310	352
12072000	Chico Creek Near Bremerton, Wash	1948-50 1962-79	473	876	1,130	1,340	1,570
12072600	Beaver Creek Near Manchester, Wash	1967-76	47	72	84	93	102
12073500	Huge Creek Near Wauna, Wash	1948-69 1978-92	127	317	448	561	689
12076500	Goldsborough Creek Near Shelton, Wash	1952-79	802	1,300	1,540	1,720	1,900

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12078400	Kennedy Creek Near Kamilche, Wash	1961-79 1992	725	1,230	1,500	1,720	1,940
12078600	Schneider Creek Trib Near Shelton, Wash	1950-69	54	88	105	116	128
12079000	Deschutes River Near Rainier, Wash	1950-82 1988-92	3,820	6,110	7,360	8,340	9,360
12080000	Deschutes River Near Olympia, Wash	1946-64	3,820	5,630	6,460	7,060	7,640
12081000	Woodland Cr Near Olympia, Wash	1950-69	91	152	183	208	232
12081300	Eaton Creek Near Yelm, Wash	1960-88	30	46	54	60	66
12082500	Nisqually River Near National, Wash	1943-92	6,230	11,700	14,600	16,800	19,100
12083000	Mineral Creek Near Mineral, Wash	1943-92	4,860	8,390	10,200	11,600	13,100
12084000	Nisqually River Near Alder, Wash	1932-44	9,940	17,500	21,800	25,200	28,800
12084500	Little Nisqually River Near Alder, Wash	1921-43	1,880	2,590	2,910	3,130	3,340
12086500	Nisqually River at La Grande, Wash	1920-31 1945-92	9,810	17,900	21,600	24,200	26,600
12087000	Mashel River Near La Grande, Wash	1941-57	2,710	4,810	6,050	7,060	8,130
12088000	Ohop Creek Near Eatonville, Wash	1928-32 1942-74	632	1,150	1,450	1,690	1,950
12088400	Nisqually River Near McKenna, Wash	1942-63 1970-79	11,400	21,100	26,000	29,600	33,300
12089500	Nisqually River at McKenna, Wash	1948-68 1978-92	9,840	19,500	24,900	29,100	33,400
12090200	Muck Creek at Roy, Wash	1957-76	403	640	748	823	896
12090400	North Fork Clover Creek Near Parkland, Wash	1960-75	148	179	191	199	206
12090500	Clover Creek Near Tillicum, Wash	1950-54 1960-70	179	361	464	546	630
12091060	Flett Creek at Mt View Memorial Pk, Wash	1968-79	45	82	102	118	135
12091100	Flett Creek at Tacoma,Wash	1980-92	57	89	104	116	127
12091200	Leach Creek Near Fircrest, Wash	1958-92	70	163	231	294	367
12091300	Leach Cr Near Steilacoom, Wash	1958-92	91	183	243	294	351
12091700	Judd Creek Near Burton, Wash	1969-79	92	144	172	194	216
12092000	Puyallup River Near Electron, Wash	1912-26 1958-92	4,410	8,250	10,300	12,000	13,600
12093000	Kapowsin Creek Near Kapowsin, Wash	1928-32 1942-70	337	590	716	808	900

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12093500	Puyallup River Near Orting, Wash	1932-92	6,600	11,400	13,700	15,400	17,000
12093900	Carbon River at Fairfax, Wash	1966-78	4,440	9,380	12,200	14,300	16,600
12094000	Carbon River Near Fairfax, Wash	1930-78 1991-92	4,210	8,280	10,600	12,400	14,300
12095000	South Prairie Creek at South Prairie, Wash	1950-79 1988-92	3,020	5,820	7,270	8,350	9,430
12096500	Puyallup River at Alderton, Wash	1916-27 1944-57	13,200	20,100	23,500	25,900	28,400
12096800	Dry Creek Near Greenwater, Wash	1957-75	21	43	58	70	83
12096950	Jim Creek Nr Greenwater, Wash	1965-75	164	254	299	332	365
12097000	White River at Greenwater, Wash	1930-78	4,750	10,200	14,000	17,200	20,800
12097500	Greenwater River at Greenwater, Wash	1930-78	1,330	3,320	4,910	6,440	8,310
12097700	Cyclone Creek Near Enumclaw, Wash	1950-72	143	294	397	487	589
12097850	White R Bl Clearwater R Near Buckley, Wash	1975-76 1983-92	11,300	21,200	26,300	30,200	34,100
12098500	White River Near Buckley, Wash	1942-92	8,880	13,200	15,100	16,400	17,500
12099600	Boise Creek at Buckley, Wash	1978-92	473	886	1,090	1,240	1,390
12100000	White River at Buckley, Wash	1978-92	7,670	15,500	19,700	23,000	26,400
12100500	White River Near Sumner, Wash	1946-73	8,120	14,300	17,200	19,200	21,200
12101500	Puyallup River at Puyallup, Wash	1942-92	22,300	38,000	45,400	50,600	55,600
12102200	Swan Creek Near Tacoma, Wash	1951-71 1990-91	117	190	230	262	294
12102800	S Fork Hylebos Creek Near Puyallup, Wash	1949-66	4.60	6.30	7.10	7.80	8.40
12103200	Joes Creek at Tacoma, Wash	1958-73	10	14	16	17	19
12103400	Green R Blw Intake Creek Near Lester, Wash	1967-77	1,190	2,900	4,110	5,170	6,390
12103500	Snow Creek Near Lester, Wash	1946-65	892	1,890	2,500	3,000	3,540
12104000	Friday Creek Near Lester, Wash	1946-77	274	656	947	1,220	1,540
12104500	Green River Near Lester, Wash	1946-90	4,720	11,500	16,100	20,100	24,600
12104700	Green Canyon Creek Near Lester, Wash	1961-74	173	305	375	428	482
12105000	Smay Creek Near Lester, Wash	1950-73	464	1,020	1,382	1,700	2,060

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12105710	North Fork Green River Near Lemolo, Wash	1957-87	1,100	2,070	2,540	2,870	3,200
12105900	Green River Below Howard Hanson Dam	1961-92	8,220	11,300	12,500	13,300	14,000
12106000	Bear Creek Near Eagle Gorge, Wash	1947-56	416	822	1,060	1,260	1,470
12106700	Green River Near Palmer, Wash	1964-92	8,400	11,600	12,900	13,800	14,600
12107200	Deep Creek Near Cumberland, Wash	1950-70	65	110	131	146	161
12108500	Newaukum Creek Near Black Diamond, Wash	1945-92	645	1,230	1,590	1,880	2,180
12112500	Big Soos Creek Near Auburn, Wash	1945-56	647	1,130	1,390	1,590	1,800
12112600	Big Soos Creek Above Hatchery, Auburn, Wash	1961-92	718	1,240	1,480	1,650	1,820
12113000	Green River Near Auburn, Wash	1962-92	8,740	11,800	13,000	13,800	14,400
12113200	Mill Creek Near Auburn, Wash	1949-70	46	72	84	93	102
12113300	Mill Creek Tributary Near Auburn, Wash	1959-75	5	9	12	14	16
12113350	Green River at Tukwila, Wash	1962-84	9,030	11,900	12,900	13,600	14,200
12113500	North Fork Cedar River Near Lester, Wash	1945-64	796	1,720	2,320	2,830	3,390
12114000	South Fork Cedar River Near Lester, Wash	1945-83	447	981	1,350	1,680	2,060
12114500	Cedar R Below Bear Cr, Near Cedar Falls, Wash	1946-64 1976-92	1,670	3,430	4,560	5,520	6,580
12115000	Cedar River Near Cedar Falls, Wash	1946-92	2,770	5,360	6,860	8,060	9,320
12115300	Green Point Creek Near Cedar Falls, Wash	1957-88	73	130	158	177	196
12115500	Rex River Near Cedar Falls, Wash	1946-92	1,700	3,190	3,970	4,560	5,160
12116100	Canyon Creek Near Cedar Falls, Wash	1946-92	48	89	110	127	144
12116500	Cedar River at Cedar Falls, Wash	1915-92	1,490	3,560	5,200	13,600	14,200
12117000	Taylor Creek Near Selleck, Wash	1957-92	891	1,810	2,360	2,810	3,300
12117500	Cedar River Near Landsburg, Wash	1905-92	2,560	5,430 0	7,460	9,270	11,40
12118500	Rock Creek Near Maple Valley, Wash	1946-76	81.8	167	211	244	276
12119000	Cedar River at Renton, Wash	1946-92	3,120	5,380	6,670	7,690	8,760
12119600	May Creek at Mouth, Near Renton, Wash	1956-58 1965-79	218	380	460	519	577

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12119700	Coal Creek Near Bellevue, Wash	1964-79	154	256	314	362	412
12119800	Valley Creek Near Bellevue, Wash	1949-67 1970-77	42	86	116	143	173
12120000	Mercer Creek Near Bellevue, Wash	1956-92	280	513	656	775	905
12120500	Juanita Creek Near Kirkland, Wash	1964-90	146	316	436	543	667
12121000	Issaquah Creek Near Issaquah, Wash	1946-64	662	1,300	1,700	2,050	2,440
12121600	Issaquah Cr At Mouth, Near Issaquah, Wash	1964-92	1,470	2,630	3,160	3,540	3,890
12121700	Tibbetts Creek Near Issaquah, Wash	1964-68 1972-77	153	310	398	466	535
12122500	Bear Creek Near Redmond, Wash	1980-81 1985-91	155	341	450	537	629
12123000	Cottage Lake Cr Near Redmond, Wash	1956-65	77.8	135	165	187	209
12123300	Evans Creek Tributary Near Redmond, Wash	1949-69	24	42	51	57	62
12124000	Evans Cr above mouth Near Redmond, Wash	1956-77	126	186	213	233	252
12125000	Sammamish River Near Redmond, Wash	1940-57	738	1,240	1,500	1,690	1,880
12125200	Sammamish River Near Woodinville, Wash	1966-92	1,470	2,100	2,350	2,520	2,670
12126000	North Creek Near Bothell, Wash	1946-74	304	445	515	566	618
12126500	Sammamish River at Bothell, Wash	1940-63	1,140	1,690	1,940	2,110	2,280
12127100	Swamp Creek at Kenmore, Wash	1964-90	440	704	838	939	1,040
12127300	Lyon Creek at Lake Forest Park, Wash	1964-75	109	147	162	174	184
12127600	McAleer Creek at Lake Forest Park, Wash	1964-75	142	215	248	273	297
12130500	S F Skykomish River Near Skykomish, Wash	1930-31 1947-70	6,410	12,600	16,300	19,300	22,500
12131000	Beckler River Near Skykomish, Wash	1930-33 1947-70	5,580	10,500	13,400	15,800	18,300
12132700	S Fk Skykomish River Trib at Baring, Wash	1951-70	107	183	220	246	272
12133000	S F Skykomish River Near Index, Wash	1914-82	23,200	44,000	55,700	64,800	74,200
12133500	Troublesome Creek Near Index, Wash	1930-41	937	2,000	2,670	3,220	3,830
12134000	North Fork Skykomish River at Index, Wash	1914-21 1930-38	13,700	24,700	30,600	35,100	39,700
12134500	Skykomish River Near Gold Bar, Wash	1929-92	39,300	72,700	90,300	104,000	117,000

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12135000	Wallace River at Gold Bar, Wash	1947-78 1989-91	2,120	3,160	3,680	4,080	4,470
12135500	Olney Creek Near Gold Bar, Wash	1947-68	972	2,080	2,860	3,560	4,360
12141000	Woods Creek Near Monroe, Wash	1947-78	1,290	2,100	2,490	2,770	3,050
12141100	Skykomish River at Monroe, Wash	1958-75	57,800	87,900	103,000	115,000	127,000
12141300	Middle Fork Snoqualmie R Near Tanner, Wash	1961-92	16,500	26,500	31,200	34,500	37,700
12141500	Middle Fork Snoqualmie Near North Bend, Wash	1912-26 1930-32	12,500	23,700	30,100	35,100	40,300
12148300	S F Tolt River Near Carnation, Wash	1983-92	1,120	2,050	2,620	3,100	3,620
12142000	N F Snoqualmie River Near Snql Falls, Wash	1930-92	7,440	12,300	14,700	16,400	18,100
12142300	Hancock Creek Near Snoqualmie, Wash	1965-79	440	734	884	998	1,110
12143000	N F Snoqualmie River Near North Bend, Wash	1909-38 1961-78	8,570	14,700	17,600	19,700	21,800
12143300	S F Snoqualmie R Trib Near North Bend, Wash	1951-70	23	41	49	55	60
12143400	S F Snoqualmie River Near Garcia, Wash	1961-92	3,900	7,160	8,810	10,000	11,300
12143600	S F Snoqualmie River at Edgewick, Wash	1984-92	6,180	10,200	12,200	13,600	14,900
12143700	Boxley Creek Near Cedar Falls, Wash	1946-92	80	158	199	230	261
12143900	Boxley Creek Near Edgewick, Wash	1982-92	125	228	285	330	376
12144000	S F Snoqualmie River at North Bend, Wash	1909-38 1946-50 1961-78 1985-92	4,990	9,060	11,200	12,800	14,400
12144500	Snoqualmie River Near Snoqualmie, Wash	1959-92	31,300	56,200	68,800	78,100	87,400
12145500	Raging River Near Fall City, Wash	1946-92	1,900	3,570	4,570	5,380	6,260
12146000	Patterson Creek Near Fall City, Wash	1948-50 1956-79	226	331	387	431	477
12147000	Griffin Creek Near Carnation, Wash	1946-79	369	668	840	977	1,120
12147500	North Fork Tolt River Near Carnation, Wash	1953-65 1968-92	4,620	7,190	8,390	9,250	10,100
12147600	South Fork Tolt River Near Index, Wash	1960-63 1968-92	1,170	1,900	2,250	2,510	2,760
12148000	South Fork Tolt River Near Carnation, Wash	1970-92	909	1,790	2,380	2,900	3,480

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12148100	S Fork Tolt River Trib Near Carnation, Wash	1955-74	124	214	261	296	331
12148500	Tolt River Near Carnation, Wash	1938-92	6,560	11,800	14,600	16,700	18,800
12149000	Snoqualmie River Near Carnation, Wash	1930-92	29,900	52,000	63,200	71,500	79,800
12150800	Snohomish River Near Monroe, Wash	1964-92	63,500	97,200	114,000	127,000	139,000
12152500	Pilchuck River Near Granite Falls, Wash	1944-80	5,100	7,760	9,010	9,900	10,800
12153000	Little Pilchuck Creek Near Lk Stevens, Wash	1947-74	265	450	544	614	684
12155500	Snohomish River at Snohomish, Wash	1942-66	56,500	86,200	101,000	113,000	124,000
12156400	Munson Creek Near Marysville, Wash	1949-69	25	40	47	53	58
12157000	Quilceda Creek Near Marysville, Wash	1947-69	163	235	270	295	320
12161000	S Fk Stillaguamish River Near Granite Falls, Wash	1929-80	16,100	24,200	27,800	30,300	32,700
12162500	S Fk Stillaguamish River Near Arlington, Wash	1938-57	19,300	26,600	29,600	31,600	33,400
12164000	Jim Creek Near Arlington, Wash	1938-69	2,780	4,130	4,740	5,180	5,600
12165000	Squire Creek Near Darrington, Wash	1951-69	2,950	4,720	5,600	6,250	6,900
12166500	Deer Creek at Oso, Wash	1918-30	7,320	9,370	10,300	10,900	11,600
12167000	N Fk Stillaguamish R Near Arlington, Wash	1929-92	22,800	32,400	36,100	38,500	40,600
12168500	Pilchuck Creen Near Bryant, Wash	1951-79	4,090	6,080	7,050	7,750	8,450
12169500	Fish Creek Near Arlington, Wash	1951-72	91	165	204	233	263
12172000	Big Beaver Creek Near Newhalem, Wash	1941-48 1963-69	2,390	4,150	5,080	5,800	6,530
12172500	Skagit River Near Newhalem, Wash	1930-39	14,100	23,300	28,500	32,700	37,100
12173500	Ruby C Below Panther C Near Newhalem, Wash	1949-56 1963-69	4,710	7,110	8,320	9,220	10,100
12174000	Ruby Creek Near Newhalem, Wash	1929-49	4,210	6,400	7,540	8,410	9,300
12175500	Thunder Creek Near Newhalem, Wash	1920-92	4,270	8,450	11,200	13,500	16,200
12176000	Thunder Creek Near Marblemount, Wash	1920-30	5,040	10,900	14,800	18,200	21,900
12177500	Stetattle Creek Near Newhalem, Wash	1934-83	2,050	4,780	6,810	8,650	10,800

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12178000	Skagit River at Newhalem, Wash	1924-92	17,900	29,200	34,500	38,200	41,900
12178100	Newhalem Creek Near Newhalem, Wash	1961-92	2,030	4,340	5,830	7,100	8,500
12179000	Skagit River Near Marblemount, Wash	1951-92	20,500	32,500	38,200	42,300	46,200
12181000	Skagit River at Marblemount, Wash	1947-57 1986-92	19,000	35,800	46,700	56,000	66,300
12181100	S Fk Cascade R Near Marblemount, Wash	1961-83	136	188	215	235	255
12181200	Salix Creek Near Marblemount, Wash	1963-83	10	18	23	27	31
12182500	Cascade River at Marblemount, Wash	1929-79	7,070	15,800	22,400	28,400	35,600
12184300	Iron Creek Near Rockport, Wash	1965-75	199	282	317	341	364
12185300	Elliot Creek Near Monte Cristo	1983-92	532	1,300	1,780	2,150	2,550
12186000	Sauk River Near Darrington, Wash	1929-92	9,300	19,300	26,000	31,900	38,500
12187500	Sauk River at Darrington, Wash	1915-32	15,500	29,800	39,500	47,900	57,400
12188300	Straight Creek Near Darrington, Wash	1965-75	295	437	503	550	596
12189000	Suiattle River Near Mansford, Wash	1939-50	9,920	18,700	24,100	28,500	33,300
12189400	Sauk River Tributary Near Darrington, Wash	1951-70 1978-88	107	179	214	239	263
12189500	Sauk River Near Sauk, Wash	1929-92	30,100	57,700	73,700	86,500	100,000
12191500	Baker River Near Concrete, Wash	1910-31 1956-59	15,700	27,400	33,100	37,200	41,300
12191800	Sulphur Creek Near Concrete, Wash	1964-76 1982	412	783	994	1,160	1,330
12193500	Baker River at Concrete, Wash	1944-92	17,800	32,300	38,500	42,600	46,300
12194000	Skagit River Near Concrete, Wash	1925-92	73,500	121,000	146,000	164,000	183,000
12196000	Alder Creek Near Hamilton, Wash	1944-79	315	577	715	819	925
12196200	Day Creek Below Day Lk, Near Lyman, Wash	1964-79	552	812	934	1,020	1,110
12196500	Day Creek Near Lyman, Wash	1944-61	4,390	5,490	5,960	6,280	6,580
12197200	Parker Creek Near Lyman, Wash	1951-70	135	187	210	226	241
12199800	East Fork Nookachamps Cr Near Big Lake, Wash	1962-78	511	639	693	731	766
12200500	Skagit River Near Mount Vernon, Wash	1941-92	66,000	105,000	127,000	144,000	161,000

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12200700	Carpenter Cr Trib Near Mount Vernon, Wash	1949-70	34	66	83	97	111
12200800	Lake Creek Near Bellingham, Wash	1949-68	118	222	281	326	374
12201500	Samish River Near Burlington, Wash	1944-83	2,590	5,330	6,950	8,250	9,630
12203500	Whatcom Creek Near Bellingham, Wash	1940-56 1968-74	601	994	1,220	1,400	1,580
12204400	Nooksack River Trib Near Glacier, Wash	1956-88	53	99	128	152	179
12205000	N Fk Nooksack River Near Glacier, Wash	1938-92	5,650	8,800	10,300	11,500	12,600
12207200	N Fk Nooksack River Near Deming, Wash	1965-75	9,680	13,900	16,000	17,500	19,000
12208000	M Fk Nooksack River Near Deming, Wash	1965-77	5,660	10,700	13,600	15,900	18,300
12209000	S F Nooksack River Near Wickersham, Wash	1925-79 1989-91	9,640	16,100	19,400	22,000	24,500
12209500	Skookum Creek Near Wickersham, Wash	1949-69	1,190	1,860	2,240	2,540	2,860
12210500	Nooksack River at Deming, Wash	1935-92	24,600	36,700	42,400	46,400	50,300
12211500	Nooksack River Near Lynden, Wash	1945-67	28,800	43,700	50,400	55,100	59,700
12212000	Fishtrap Creek at Lynden, Wash	1949-74	358	519	589	638	685
12212700	Tenmile Creek Tributary Near Bellingham, Wash	1949-67	25	46	57	66	75
12212800	Tenmile Creek Trib #2 Near Bellingham, Wash	1956-87	23	46	59	68	78
12213100	Nooksack River at Ferndale, Wash	1950-92	24,600	38,500	46,400	52,800	59,500
12323000	Columbia River at Birchbank, BC	1969-92	145,000	197,000	222,200	241,000	259,000
12395500	Pend Oreille River at Newport, ID	1953-92	82,600	124,000	140,000	149,000	157,000
12395800	Deer Creek Near Dalkena, Wash	1954-73	45	71	83	91	98
12395900	Davis Creek Near Dalkena, Wash	1954-73	88	139	163	180	198
12396000	Calispell Creek Near Dalkena, Wash	1951-92	508	1,070	1,450	1,760	2,120
12396450	Little Muddy Creek at Ione, Wash	1954-73	92	200	265	317	372
12396500	Pend Oreille River Near Ione, Wash	1953-92	85,300	124,000	138,000	147,000	154,000
12396900	Sullivan Creek Near Metaline Falls, Wash	1958-74	1,040	1,600	1,900	2,120	2,340
12397100	Outlet Creek Near Metaline Falls, Wash	1959-92	467	723	852	919	1,050

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12398000	Sullivan Creek at Metaline Falls, Wash	1954-68	1,300	2,410	3,000	3,450	3,910
12398500	Pend Oreille River Near Metaline Falls, Wash	1952-64	99,100	122,000	131,000	136,000	142,000
12398600	Pend Oreille River at International Boundary	1963-92	83,500	131,000	150,000	162,000	172,000
12399500 Columbia	River at International Boundary	1958-92	235,000	377,000	450,000	504,000	560,000
12400500	Sheep Creek Near Northport, Wash	1930-42	1,720	2,510	2,850	3,090	3,310
12401500	Kettle River Near Ferry, Wash	1929-92	12,300	16,300	18,000	19,000	20,000
12403700	Third Creek Near Curlew, Wash	1954-73	9	18	22	25	28
12404500	Kettle River Near Laurier, Wash	1930-92	21,000	27,800	30,600	32,500	34,200
12405400	Nancy Creek Near Kettle Falls, Wash	1954-72	54	126	170	207	246
12407500	Sheep Creek at Springdale, Wash	1953-72	44	78	94	105	116
12407520	Deer Creek Near Valley, Wash	1960-79	119	285	391	498	576
12407600	Thomason Creek Near Chewelah, Wash	1954-73	6	10	12	14	16
12407700	Chewelah Creek at Chewelah, Wash	1957-74	165	318	400	463	527
12408200	Patchen (Bighorn) Cr Near Tiger, Wash	1954-73	9	21	27	32	37
12408300	Little Pend Oreille River Near Colville, Wash	1958-79	301	688	906	1,070	1,240
12408400	Narcisse Creek Near Colville, Wash	1954-73	29	58	76	89	104
12408420	Haller Creek Near Arden, Wash	1960-79	41	118	172	221	276
12408500	Mill Creek Near Colville, Wash	1940-86	298	562	690	782	870
12409000	Colville River at Kettle Falls, Wash	1923-92	1,130	2,260	2,810	3,200	3,570
12409500	Hall Creek at Inchelium, Wash	1913-29 1972-73	416	1,060	1,460	1,770	2,100
12410000	Stranger Creek at Meteor, Wash	1917-29	56	226	358	476	608
12410600	S Fork Harvey Creek Near Cedonia, Wash	1954-73	22	36	41	45	49
12410650	N Fork Harvey Creek Near Cedonia, Wash	1954-73	6	11	13	14	16
12419000	Spokane River Near Post Falls, Id	1913-92	26,000	37,000	41,100	43,800	46,200
12419500	Spokane River Near Otis Orchard, Wash	1951-83	27,100	37,100	41,800	45,100	48,400

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
12422500	Spokane River at Spokane, Wash	1891-1992	24,900	36,600	41,300	44,400	47,200
12423550	Hangman Creek Trib Near Latah, Wash	1961-76	55	171	240	293	346
12423700	S F Rock Cr Tributary Near Fairfield, Wash	1962-76	25	37	42	45	48
12423900	Stevens Creek Tributary Near Moran, Wash	1954-73	18	67	103	133	166
12424000	Hangman Creek at Spokane, Wash	1948-92	6,530	13,100	16,600	19,300	22,000
12427000	Little Spokane River at Elk, Wash	1949-79	110	150	169	182	195
12429200	Bear Creek Near Milan, Wash	1963-75	50	82	98	109	120
12429600	Deer Creek Near Chattaroy, Wash	1962-75	138	232	284	325	368
12429800	Mud Creek Near Deer Park, Wash	1954-73	12	23	28	32	36
12430370	Bigelow Gulch Near Spokane, Wash	1962-75	23	126	252	400	614
12431000	Little Spokane River at Dartford, Wash	1929-32 1947-92	1,300	2,260	2,710	3,020	3,320
12431100	Little Creek at Dartford, Wash	1963-77	39	226	422	629	898
12433000	Spokane River at Long Lake, Wash	1939-92	30,900	44,000	49,100	52,500	55,600
12433200	Chamokane Creek Near Long Lk, Wash	1971-79 1988-92	355	1,160	1,830	2,480	3,270
12433300	Spring Cr Tributary Near Reardan, Wash	1954-73	48	101	131	155	180
12433500	Spokane River Near Long Lk, Wash	1913-41	28,200	39,600	44,300	47,500	50,500
12433580	Cottonwood Creek at Davenport, Wash	1963-77	224	1,270	2,210	3,090	4,110
12433800	Granite Creek Near Republic, Wash	1954-73	12	24	32	38	44
12434500	Sanpoil River Near Keller, Wash	1953-59 1972-79	1,510	3,160	4,210	5,090	6,060
12435000	Sanpoil River at Keller, Wash	1911-17 1953-59	1,470	2,570	3,180	3,660	4,160
12436500	Columbia River at Grand Coulee, Wash	1928-92	298,000	474,000	556,000	615,000	672,000
12437500	Nespelem River at Nespelem, Wash	1911-29	171	498	706	875	1,050
12437930	East Fork Foster Creek at Leahy, Wash	1963-77	73	260	398	519	654
12437950	East Fork Foster C Trib Near Bridgeport, Wash	1957-77	24	107	192	283	404
12437960	West Fork Foster Creek Near Bridgeport, Wash	1963-77	60	178	266	349	445

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12438000	Columbia River at Bridgeport, Wash	1952-92	261,000	418,000	497,000	555,000	614,000
12438700	Okanogan River Near Oliver, BC	1962-92	1,690	3,190	3,940	4,490	5,030
12439200	Dry Creek Trib Near Molson, Wash	1958-77	10	54	93	129	170
12439300	Tonasket Creek at Oroville, Wash	1967-91	59	254	438	626	864
12439500	Okanogan River at Oroville, Wash	1943-92	1,840	3,170	3,750	4,140	4,510
12441700	Middle Fk Toats Coulee Cr Near Loomis, Wash	1965-75	206	490	667	812	967
12442000	Toats Coulee Creek Near Loomis, Wash	1920-26 1957-79	523	1,240	1,750	2,210	2,730
12442500	Similkameen River Near Nighthawk, Wash	1911-92	16,300	26,200	30,700	34,000	37,100
12443500	Similkameen River Near Oroville, Wash	1911-28	16,800	21,800	23,600	24,800	25,800
12443700	Spectable Lake Trib Near Loomis, Wash	1961-76	4	54	131	226	364
12444100	Whitestone Creek Near Tonasket, Wash	1959-72	9	15	20	23	26
12444400	Siwash Creek Trib Near Tonasket, Wash	1959-77	7	31	50	68	89
12445000	Okanogan River Near Tonasket, Wash	1929-92	16,100	27,200	32,600	36,600	40,600
12445800	Omak Creek Trib Near Distaul, Wash	1955-75	6	13	16	19	21
12446000	Okanogan River at Okanogan, Wash	1911-25	16,900	21,400	23,100	24,200	25,100
12446500	Salmon Creek Near Conconully, Wash	1911-22	160	350	479	592	720
12447200	Okanogan River at Malott, Wash	1959-92	16,300	26,300	31,600	35,600	39,700
12447300	Okanogan River Near Malott, Wash	1958-67	16,600	24,000	27,200	29,400	31,500
12447380	Pine Creek Near Mazama, Wash	1966-88	158	294	371	431	494
12447390	Andrews Creek Near Mazama, Wash	1969-92	381	702	902	1,070	1,250
12447400	Doe Creek Near Winthrop, Wash	1957-75	24	56	75	89	104
12447430	Ortell Creek Near Winthrop, Wash	1965-75	58	90	106	117	127
12448700	Williams Creek Near Twisp, Wash	1965-75	59	86	99	108	118
12448900	Little Bridge Creek Near Twisp, Wash	1965-75	132	232	281	318	354
12449500	Methow River at Twisp, Wash	1920-29 1934-62	11,100	18,800	22,200	24,500	26,700

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12449600	Beaver Creek Blw South Fork, Near Twisp, Wash	1960-79	133	374	533	665 8	808
12449790	Rainy Creek Near Methow, Wash	1965-75	54.6	112	148	178	211
12449950	Methow River Near Pateros, Wash	1959-92	11,900	21,100	25,900	29,600	33,300
12450500	Methow River at Pateros, Wash	1904-20 1959-74	12,200	22,300	28,300	33,100	38,200
12450700	Columbia River Below Wells Dam, Wash	1968-92	227,000	324,000	370,000	405,000	438,000
12451000	Stehekin River at Stehekin, Wash	1927-92	9,550	14,200	16,200	17,700	19,200
12451500	Railroad Creek at Lucerne, Wash	1927-57	1,280	2,190	2,720	3,150	3,600
12452500	Chelan River at Chelan, Wash	1904-92	8,150	13,600	15,800	17,100	183,000
12452800	Entiat River Near Ardenvoir, Wash	1958-92	2,700	4,460	5,350	6,020	6,690
12452880	Tillicum Cr Near Ardenvoir, Wash	1965-75	32	67	87	103	120
12453000	Entiat River at Entiat, Wash	1911-25 1952-58	3,360	5,090	5,980	6,660	7,340
12453700	Columbia R at Rocky Reach Dam, Wash	1961-92	260,000	412,000	489,000	548,000	607,000
12454000	White River Near Plain, Wash	1955-83	4,740	7,490	9,110	10,400	11,800
12454290	Little Wenatchee R Trib Near Telma, Wash	1965-75	96	132	149	162	174
12455000	Wenatchee River Below Wenatchee Lk, Wash	1932-79	7,040	10,000	11,400	12,400	13,400
12456300	Brush Cr Near Telma, Wash	1965-75	66	158	223	280	346
12456500	Chiwawa River Near Plain, Wash	1937-49 1991-92	2,960	4,820	5,720	6,380	7,030
12457000	Wenatchee River at Plain, Wash	1911-79 1990-92	11,700	17,900	20,900	23,100	25,300
12457300	Skinney Creek at Winton, Wash	1954-73	28	55	69	81	93
12457900	Chatter Creek Near Leavenworth, Wash	1966-75	55	87	103	115	127
12458000	Icicle Creek Near Leavenworth, Wash	1912-14 1937-79	4,510	7,550	9,120	10,300	11,500
12458900	Posey Canyon Near Leavenworth, Wash	1954-73	2	9	15	20	26
12459000	Wenatchee River at Peshastin, Wash	1929-92	16,100	24,100	28,000	30,700	33,400
12459400	Trosen Creek Near Peshastin, Wash	1960-75	27	62	82	98	114
12461000	Wenatchee River at Dryden, Wash	1905-06 1910-18	18,200	29,100	34,600	38,700	42,800

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12461100	East Branch Mission Cr Near Cashmere, Wash	1955-74	22	61	91	118	150
12461200	E Br Mission Cr Trib, Near Cashmere, Wash	1955-88	7	21	30	37	45
12461400	Mission Creek Near Cashmere, Wash	1959-79	193	664	1,110	1,570	2,170
12461500	Sand Creek Near Cashmere, Wash	1954-73	64	158	228	290	363
12462000	Mission Creek at Cashmere, Wash	1954-73	188	376	502	609	730
12462500	Wenatchee River at Monitor, Wash	1963-92	17,400	26,300	30,900	34,500	38,100
12462600	Columbia River Below Rock Island Dam, Wash	1961-92	253,000	415,000	503,000	571,000	641,000
12462700	Moses Creek at Waterville, Wash	1954-73	9	161	397	681	1,080
12462800	Moses Creek at Douglas, Wash	1955-76	47	275	473	654	860
12463000	Douglas Creek Near Alstown, Wash	1950-60 1963-68	359	5,350	12,600	21,300	32,200
12463600	Rattlesnake Creek Trib Near Soap Lake, Wash	1961-77	9	70	147	238	366
12463700	McCarteney Creek Trib Near Farmer, Wash	1962-76	5	45	95	152	229
12463800	Pine Canyon Tributary Near Farmer, Wash	1962-76	2	33	79	136	216
12464500	Columbia River at Trinidad, Wash	1938-63	381,000	539,000	604,000	648,000	688,000
12464600	Schnebly Coulee Trib Near Vantage, Wash	1955-74	7	44	82	122	173
12464650	South Fork Crab Creek Trib at Waukon, Wash	1954-73	18	50	73	93	117
12465000	Crab Creek at Irby, Wash	1943-92	846	4,650	7,700	10,300	13,200
12465100	Connawai Creek Trib Near Govan, Wash	1958-77	5	28	52	77	110
12465300	Broadax Draw Tributary Near Wilbur, Wash	1955-74	23	86	134	176	224
12465400	Wilson Creek Near Almira, Wash	1969-79	967	3,460	5,320	6,940	8,760
12465500	Wilson Creek at Wilson Creek, Wash	1951-79	883	4,450	7,630	10,700	14,300
12467000	Crab Creek at Wilson Creek, Wash	1943-92	284	1,660	3,280	5,140	7,760
12467400	Haynes Canyon Near Coulee City, Wash	1959-76	6	37	68	100	140
12468500	Park Creek Near Coulee City, Wash	1946-68	24	37	42	46	50
12470300	Iron Springs Creek Near Winchester, Wash	1959-76	17	64	100	133	170

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12470500	Rocky Ford Creek Near Ephrata, Wash	1943-91	97	133	151	164	178
12471100	Paha Coulee Tributary Near Ritzville, Wash	1962-76	114	245	317	371	427
12471200	Lind Coulee Tributary Near Lind, Wash	1961-77	3	55	132	221	338
12471270	Farrier Coulee Near Schrag, Wash	1963-76 1978-79	152	1,070	2,020	2,970	4,140
12471300	Weber Coulee Tributary Near Ruff, Wash	1959-72	2	36	96	144	290
12471500	Crab Creek Near Warden, Wash	1943-53 1956-65	118	971	2,180	3,730	6,080
12472500	Crab Creek Near Smyrna, Wash	1943-59	134	865	1,840	3,060	4,920
12472600	Crab Creek Near Beverly, Wash	1960-92	298	506	599	664	725
12472800	Columbia River Below Priest Rapids Dam, Wash	1917-37 1938-92	331,000	481,000	552,000	604,000	654,000
12474500	Yakima River Near Martin, Wash	1904-78	1,480	2,850	3,810	4,680	5,680
12474700	Mosquito Creek Near Easton, Wash	1968-77	82.5	144	177	203	230
12476000	Kachess River Near Easton, Wash	1904-78	1,290	1,910	2,230	2,460	2,700
12477000	Yakima River at Easton, Wash	1910-15 1941-54	3,150	6,340	8,380	10,100	12,000
12479000	CleElum River Near Rosyln, Wash	1904-78	4,390	7,830	10,100	12,000	14,200
12479500	Yakima River at Cle Elum, Wash	1907-78 1988-90	6,820	12,900	16,800	19,900	23,400
12480700	Hovey Creek Near Cle Elum, Wash	1955-74	31	51	61	67	74
12483300	South Fk Manastash Cr Trib Near Ellensburg, Wash	1955-74	32	66	88	106	125
12483800	Naneum Creek Near Ellensburg, Wash	1957-78	412	733	905	1,040	1,170
12484200	Johnson Canyon Trib Near Kittitas, Wash	1956-75	2	29	69	116	181
12484500	Yakima River at Umtanum, Wash	1907-17 1925-72	8,630	17,200	22,700	27,400	32,600
12484600	McPherson Canyon at Wymer, Wash	1955-77	29	181	309	418	537
12485900	Pine Canyon Near Naches, Wash	1961-76	12	48	84	124	178
12487400	Deep Creek Near Goose Prairie, Wash	1966-75	464	742	876	972	1,070
12488000	Bumping River Near Nile, Wash	1914-78	1,380	2,400	3,000	3,490	4,020

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12488300	American River Trib Near Nile, Wash	1955-74	17	28	33	36	40
12488500	American River Near Nile, Wash	1940-92	1,470	2,500	3,050	3,470	3,890
12489500	Naches River Near Nile, Wash	1905-17	6,300	11,900	15,500	18,600	21,900
12491500	Tieton River at Tieton Dam Near Naches, Wash	1925-78	2,070	3,620	4,630	5,480	6,440
12491700	Hause Creek Near Rimrock, Wash	1955-88	26	63	89	113	140
12492500	Tieton River Near Naches, Wash	1926-78	1,860	3,530	4,620	5,560	6,610
12494000	Naches River Near Naches, Wash	1916-79	7,000	14,000	17,700	20,500	23,400
12499000	Naches River Near North Yakima, Wash	1900-10 1987-90	8,220	17,700	24,000	29,400	35,400
12500450	Yakima R at Union Gap, Wash	1967-92	13,900	28,500	36,700	43,000	49,600
12500500	North Fork Ahtanum Cr Near Tampico, Wash	1910-21 1932-79	381	712	905	1,060	1,230
12501000	S F Autanum Cr Near Tampico, Wash	1915-24 1931-78	97	268	402	529	681
12502000	Ahtanum Cr Near Tampico, Wash	1909-13 1960-68	538	1,210	1,680	2,100	2,580
12502500	Ahtanum Creek at Union Gap, Wash	1960-92	414	1,060	1,470	1,810	2,170
12505000	Yakima River Near Parker, Wash	1908-78	13,800	29,600	38,800	46,100	53,700
12506000	Toppenish Creek Near Fort Simcoe, Wash	1910-24	696	1,460	1,930	2,310	2,730
12506500	Simcoe Cr Near Fort Simcoe, Wash	1909-23	241	850	1,340	1,790	2,330
12507600	Shinando Creek Trib Near Goldendale, Wash	1955-74	4	16	28	40	54
12507660	Satus Creek Tributary Near Toppenish, Wash	1963-77	153	664	1,170	1,690	2,380
12508500	Satus Creek Nearr Toppenish, Wash	1914-24	1,410	3,260	4,450	5,450	6,540
12508990	Yakima River at Mabton, Wash	1971-92	12,900	30,100	40,200	48,200	56,500
12509500	Yakima River Near Prosser, Wash	1914-18 1920-33	14,300	34,200	48,400	61,200	76,100
12509800	Snipes Cr Tributary Near Benton City, Wash	1967-77	22	255	601	1,030	1,660
12510500	Yakima River at Kiona, Wash	1934-92	14,200	30,200	39,900	47,900	56,400
12512550	Providence Coulee Near Cunningham, Wash	1978-91	95	1,480	3,450	5,710	8,740

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12512600	Hatton Coulee Trib No 2 Near Cunningham, Wash	1961-76	3	70	202	393	704
12512700	Hatton Coulee Tributary Near Hatton, Wash	1956-75	4	53	126	217	351
12513000	Esquatzel Coulee at Connell, Wash	1953-92	46	2,250	8,640	18,000	33,400
13334300	Snake River Near Anatone, Wash	1959-92R	98,500	156,000	182,000	200,000	218,000
13334500	Asotin Creek Near Asotin, Wash	1929-59	338	715	952	1,150	1,360
13334700	Asotin Creek Near Asotin, Wash	1960-82 1990-92	421	1,460	2,380	3,300	4,460
13335200	Critchfield Draw Near Clarkston, Wash	1959-76	15	252	620	1,080	1,730
13343450	Dry Creek at Mouth Near Clarkston, Wash	1963-77	79	496	1,060	1,790	2,900
13343500	Snake River Near Clarkston, Wash	1916-22 1929-72	192,000	279,000	316,000	341,000	365,000
13343520	Clayton Gulch Near Alpowa, Wash	1961-76	100	261	361	442	527
13343620	S F Deadman Creek Trib Near Pataha, Wash	1961-76	17	157	305	452	628
13343660	Smith Gulch Trib Near Pataha, Wash	1955-74	49	240	409	569	760
13343800	Meadow Creek Near Central Ferry, Wash	1964-78	676	1,830	2,550	3,110	3,710
13344500	Tucannon River Near Starbuck, Wash	1959-90	1,490	4,530	6,670	8,500	10,500
13346100	Palouse River at Colfax, Wash	1956-79	4,610	8,390	10,400	11,900	13,400
13348000	South Fork Palouse River at Pullman, Wash	1934-42 1959-81	1,050	2,510	3,520	4,400	5,410
13348400	Missouri Flat Creek Trib Near Pullman, Wash	1955-74	36	110	169	225	292
13348500	Missouri Flat Creek at Pullman, Wash	1935-40 1960-79	401	848	1,150	1,400	1,690
13349210	Palouse River at Colfax, Wash	1963-92	5,990	12,200	15,700	18,500	21,400
13349300	Palouse River Trib at Colfax, Wash	1955-88	29	98	152	202	260
13349309	Palouse River Trib at Winona, Wash	1967-77	32	98	140	175	210
13349350	Hardman Draw Trib at Plaza, Wash	1955-74	33	117	203	298	427
13349400	Pine Creek at Pine City, Wash	1962-79	1,970	5,780	8,530	11,000	13,700
13349500	Rock Creek Near Ewan, Wash	1915-17 1965-75	1,140	3,680	5,280	6,540	7,840

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13349670	Pleasant Valley Cr Trib Near Thornton, Wash	1967-77	18	54	68	77	87
13349800	Imbler Cr Tributary Near Lamont, Wash	1967-77	59	124	166	202	240
13350500	Union Flat Creek Near Colfax, Wash	1954-79	889	2,050	2,720	3,260	3,820
13350800	Willow Cr Trib Near Lacrosse, Wash	1967-77	18	45	61	73	85
13351000	Palouse River at Hooper, Wash	1901-16 1951-92	7,850	18,000	24,400	29,700	35,400
13352200	Cow Creek Trib Near Ritzville, Wash	1951 1955-73	22	100	167	229	302
13352500	Cow Creek at Hooper, Wash	1962-79	123	603	1,070	1,550	2,160
13352550	Stewart Canyon Trib Near Riparia, Wash	1958-75	21	119	225	341	496
13353000	Snake R Below Ice Harbor Dam, Wash	1910-16 1963-90	191,000	281,000	319,000	346,000	371,000
14013000	Mill Creek Near Walla Walla, Wash	1914-17 1940-92	880	1,880	2,500	3,010	3,570
14013500	Blue Creek Near Walla Walla, Wash	1940-42 1944-71	324	719	947	1,130	1,310
14014000	Yellowhawk Cr at Walla Walla, Wash	1942-52	173	314	385	436	488
14014500	Garrison Cr at Walla Walla, Wash	1943-52	33	57	70	79	88
14015000	Mill Creek at Walla Walla, Wash	1942-92	1,000	1,850	2,300	2,650	3,000
14015900	Spring Creek Trib Near Walla Walla, Wash	1955-74	23	140	277	435	654
14016000	Dry Creek Near Walla Walla, Wash	1949-67	548	1,660	2,430	3,080	3,810
14016500	East Fk Touchet R Near Dayton, Wash	1944-51 1956-68	862	2,050	2,880	3,620	4,460
14016600	Hatley Creek Near Dayton, Wash	1955-74	77	267	396	502	614
14016650	Davis Hollow Near Dayton, Wash	1956-75	9	86	190	317	498
14017000	Touchet River at Bolles, Wash	1952-89	2,640	5,420	7,000	8,240	9,520
14017040	Thorn Hollow Near Dayton, Wash	1962-76	35	182	308	424	558
14017070	East Fork McKay Creek Near Huntsville, Wash	1963-77	59	366	717	1,110	1,640
14017200	Badger Hollow Near Clyde, Wash	1955-74	48	318	592	868	1,210
14017500	Touchet River Near Touchet, Wash	1946-53 1955-59	3,780	8,960	12,300	15,100	18,100
14018500	Walla Walla River Near Touchet, Wash	1952-92	6,280	14,800	20,200	24,600	29,400

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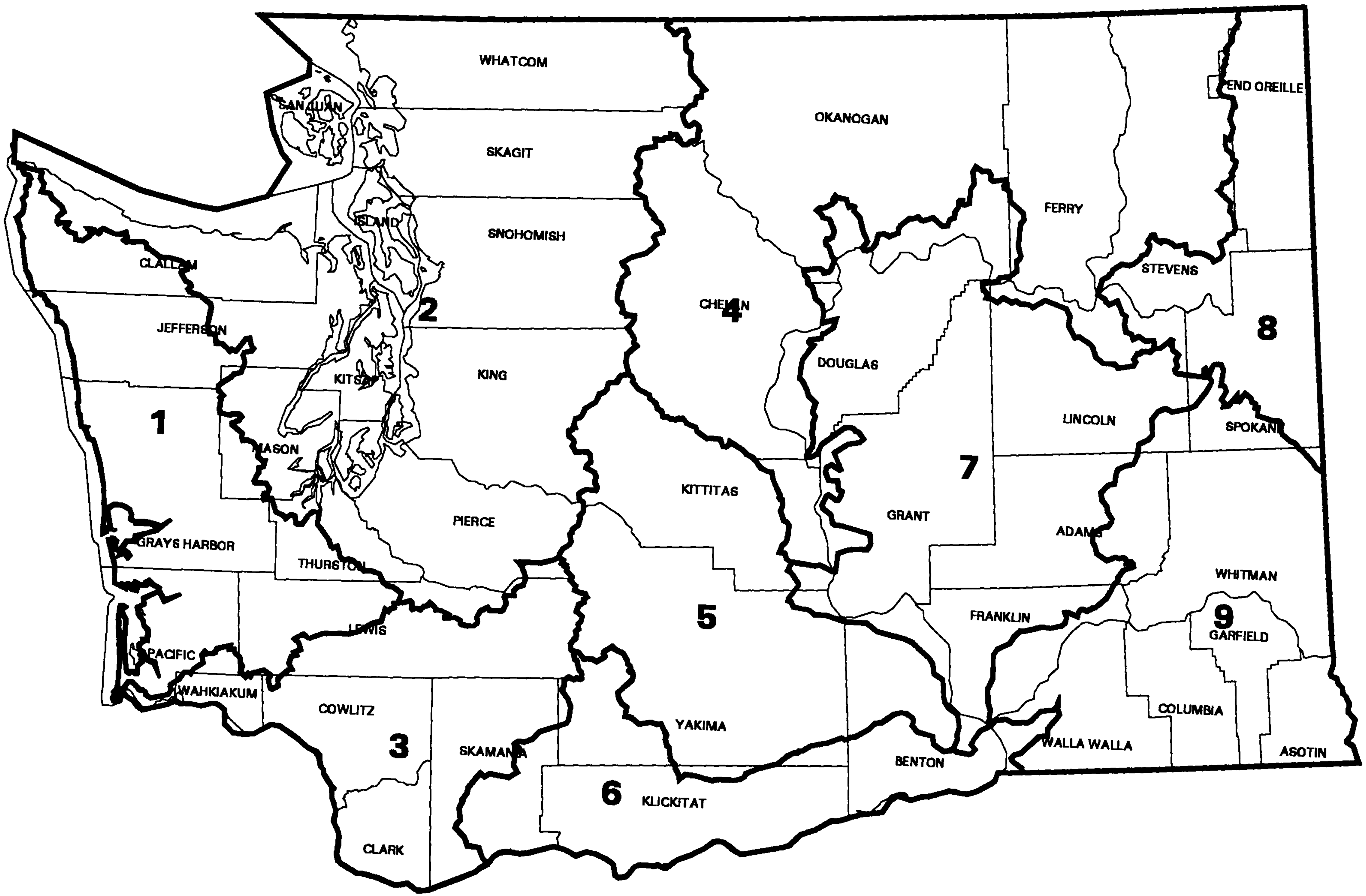
14034250	Glade Creek Trib Near Bickleton, Wash	1961-76	7	25	38	50	64
14034325	Alder Creek Near Bickleton, Wash	1963-77	207	693	1,110	1,520	2,030
14107000	Klickitat R Abv W Fork Near Glenwood, Wash	1945-78	1,860	3,040	3,670	4,160	4,650
14110000	Klickitat River Near Glenwood, Wash	1910-79	3,180	5,500	6,820	7,870	8,970
14111800	W F Little Klickitat R Near Goldendale, Wash	1961-75	105	302	459	605	781
14112000	Little Klickitat R Near Goldendale, Wash	1947-50 1958-78	1,070	3,060	4,560	5,940	7,550
14112200	Little Klickitat R Trib Near Goldendale, Wash	1960-88	25	88	148	208	287
14112400	Mill Creek Near Blockhouse, Wash	1965-78	113	250	331	397	466
14112500	Little Klickitat R Near Wahkiacus, Wash	1945-81	3,260	9,250	13,200	16,400	19,900
14113000	Klikitat River Near Pitt, Wash	1929-92	7,860	19,200	26,800	33,300	40,600
14121300	White Salmon R Below Cascades Cr, Wash	1958-78	700	1,150	1,390	1,560	1,740
14121500	Trout Lake Creek Near Trout Lake, Wash	1960-69	1,590	2,610	3,140	3,540	3,940
14122000	White Salmon River Near Trout Lake, Wash	1958-67	1,970	3,380	4,150	4,750	5,360
14123000	White Salmon River at Husum, Wash	1910-18 1930-41	2,830	5,510	7,230	8,700	10,300
14123500	White Salmon R Near Underwood, Wash	1916-30 1936-92	4,650	8,540	10,600	12,300	14,000
14124500	Little White Salmon River at Willard, Wash	1945-61	2,780	3,780	4,190	4,480	4,740
14125000	Little White Salmon R at Willard, Wash	1950-63	2,520	3,600	4,150	4,560	5,000
14125200	Rock Creek Near Willard, Wash	1949-68	192	345	435	508	585
14125500	Little White Salmon River Near Cook, Wash	1957-77	3,300	6,770	9,040	11,000	13,200
14126300	Columbia River Trib at Home Valley, Wash	1950-70	44	75	91	104	116
14127000	Wind R Abv Trout Creek Near Carson, Wash	1945-69	5,240	7,780	8,950	9,790	10,600
14127200	Layout Cr Near Carson, Wash	1966-75	342	578	696	783	871
14128500	Wind River Near Carson, Wash	1935-79	13,800	25,900	32,400	37,600	42,800
14143200	Canyon Creek Near Washougal, Wash	1949-70	128	220	274	317	364
14144000	Little Washougal River Near Washougal, Wash	1952-68	1,250	2,020	2,420	2,720	3,010

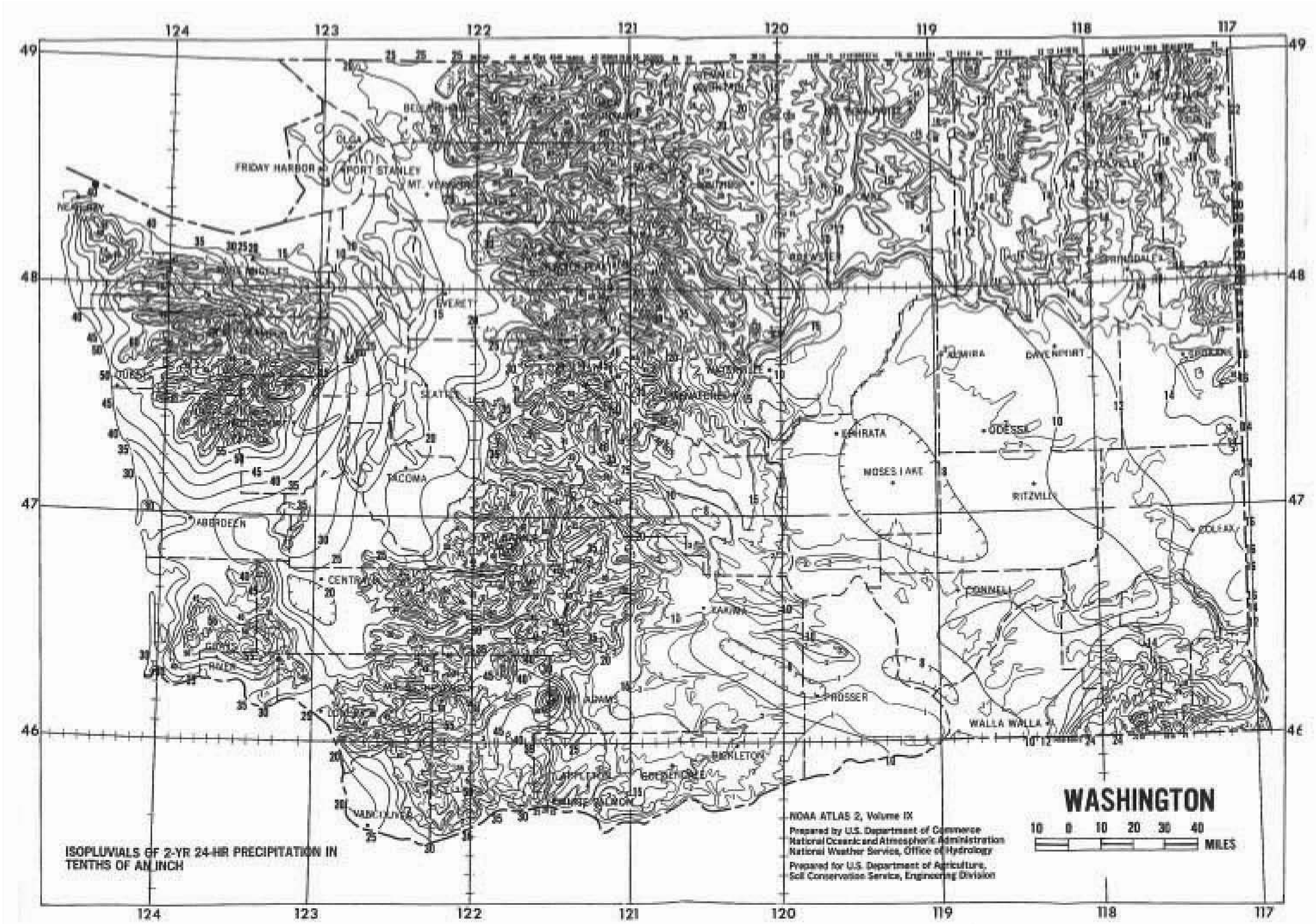
Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
14144550	Shanghai Creek Near Hockinson, Wash	1950-70	76	128	154	174	193
14144600	Groeneveld Creek Near Camas, Wash	1958-81	42	68	82	94	106
14211900	Burnt Bridge Creek at Vancouver, Wash	1949-71	77	130	159	182	207
14212000	Salmon Creek Near Battle Ground, Wash	1944-79	835	1,430	1,750	2,000	2,260
14213200	Lewis River Near Trout Lake, Wash	1959-72	5,900	11,100	14,200	16,700	19,400
14213500	Big Creek Near Trout Lk, Wash	1956-79	398	759	960	1,120	1,280
14214000	Rush Creek Near Trout Lk, Wash	1956-65	471	952	1,220	1,430	1,650
14214500	Meadow Creek Near Trout Lk, Wash	1956-65	278	447	539	610	684
14215000	Rush Creek Near Cougar, Wash	1956-62 1964-74	720	1,390	1,810	2,170	2,560
14215500	Curley Creek Near Cougar, Wash	1956-74	582	1,450	2,040	2,550	3,130
14216000	Lewis River Above Muddy River, Wash	1928-34 1955-75	9,310	19,500	25,200	29,600	34,200
14216500	Muddy River Near Cougar, Wash	1955-73 1984-92	6,420	11,500	14,400	16,700	19,000
14216800	Pine Creek Near Cougar, Wash	1958-70	978	1,590	1,900	2,150	2,390
14218000	Lewis River Near Cougar, Wash	1959-78	15,800	51,800	78,100	101,000	127,000
14218300	Dog Creek at Cougar, Wash	1958-74	311	507	616	702	792
14219000	Canyon Creek Near Amboy, Wash	1923-34	5,910	10,400	12,900	14,900	16,900
14219500	Lewis River Near Amboy, Wash	1912-31	33,600	62,200	77,000	88,100	99,200
14219800	Speelyai Creek Near Cougar, Wash	1960-92	1,710	3,000	3,580	4,020	4,460
14220500	Lewis River at Ariel, Wash	1931-92	30,900	62,600	80,500	94,600	109,000
14221500	Cedar Creek Near Ariel, Wash	1952-55 1962-69	1,480	2,150	2,490	2,750	3,010
14222500	East Fork Lewis River Near Heisson, Wash	1930-92	9,000	14,100	16,500	18,200	20,000
14222700	East Fork Lewis R Trib Near Woodland, Wash	1950-67	36	77	105	130	159
14223000	Kalama River Near Kalama, Wash	1917-45	7,790	14,600	19,200	23,200	27,700
14223500	Kalama R Below Italian Cr Near Kalama, Wash	1947-79	10,400	15,100	17,500	19,300	21,100
14223800	Columbia River Trib at Carrolls, Wash	1950-70	52	82	98	110	123

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
14224500	Clear Fk Cowlitz River Near Packwood, Wash	1908-17 1931-41	1,720	3,620	4,940	6,110	7,460
14225500	Lake Creek Near Packwood, Wash	1964-80	150	827	1,580	2,420	3,570
14226000	Lake Creek at Mouth, Near Packwood, Wash	1964-77	299	945	1,450	1,910	2,460
14226500	Cowlitz River at Packwood, Wash	1912-20 1930-92	13,800	25,400	32,100	37,400	43,000
14226800	Skate Creek Trib Near Packwood, Wash	1959-77	60	116	151	181	213
14226900	Skate CreekTrib No 2 Near Packwood, Wash	1959-75 1978-88	109	220	298	367	446
14230000	Johnson Creek Near Packwood, Wash	1908-14 1919-24	1,360	2,590	3,310	3,890	4,510
14231100	Miller Creek at Randle, Wash	1950-70	81	113	129	140	151
14231700	Chamber Creek Near Packwood, Wash	1966-75	227	349	416	468	522
14232000	Niggerhead Creek Near Randle, Wash	1951-63	2,610	3,860	4,420	4,830	5,220
14232500	Cispus River Near Randle, Wash	1930-92	8,160	15,100	18,800	21,600	24,000
14233200	Quartz Creek Near Kosmos, Wash	1965-75	90	147	178	203	228
14233400	Cowlitz River Near Randle, Wash	1949-92	28,800	51,700	65,300	76,400	88,200
14233500	Cowlitz River Near Kosmos, Wash	1949-68	26,200	38,300	44,800	49,700	54,700
14235000	Cowlitz River at Mossyrock, Wash	1927-35 1948-60	27,600	46,600	58,200	67,800	78,300
14235300	Tilton River Near Mineral, Wash	1950-70	86	124	140	152	163
14235500	West Fork Tilton River Near Morton, Wash	1951-79	2,110	3,640	4,560	5,310	6,130
14236200	Tilton R Above Bear Canyon Creek, Wash	1957-92	10,800	18,600	22,100	24,500	26,800
14236500	Tilton River Near Cinebar, Wash	1942-59	10,900	18,200	22,200	25,400	28,600
14237000	Klickitat Creek at Mossyrock, Wash	1949-72	104	161	187	205	222
14237500	Winston Creek Near Silver Lake, Wash	1950-77	1,200	2,080	2,580	2,980	3,410
14238000	Cowlitz River Below Mayfield Dam, Wash	1963-92	26,400	46,800	57,900	66,500	75,400
14239000	Salmon Creek Near Toledo, Wash	1962-79	3,610	6,940	8,660	9,940	11,200
14239100	North Fork Lacamas Creek Near Ethel, Wash	1950-69	24	34	39	42	45
14239700	Olequa Creek Tributary Near Winlock, Wash	1950-69	22	37	45	52	58

Station Number	Station Name	Period of Record	Q2 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)
14240800	Green River Near Kid Valley, Wash	1981-92	6,270	11,100	13,800	15,800	18,000
14241100	N F Toutle River at Kid Valley, Wash	1980-92	11,300	24,000	31,200	36,800	42,600
14241490	S. F. Toutle R at Camp 12 Near Toutle, Wash	1981-92	8,140	16,000	20,300	23,600	27,000
14241500	South Fork Toutle River at Toutle, Wash	1940-57	6,280	9,610	11,400	12,900	14,400
14242500	Toutle River Near Silver Lake, Wash	1920-23 1930-79	17,100	28,200	34,700	39,900	45,500
14242580	Toutle River at Tower Rd Near Silver Lk, Wash	1982-92	20,200	40,600	51,500	59,800	68,200
14242600	Toutle R Trib Nr Castle Rock, Wash	1950-70	39	76	98	116	135
14243000	Cowlitz River at Castle Rock, Wash	1963-92	55,000	90,500	107,000	118,000	128,000
14243500	Delameter Creek Near Castle Rock, Wash	1950-69	1,260	2,120	2,570	2,920	3,290
14245000	Coweman River Near Kelso, Wash	1950-84	4,910	7,600	9,030	10,100	11,300
14247500	Elochoman River Near Cathlamet, Wash	1941-79	4,900	7,400	8,590	9,450	10,300
14248100	Risk Creek Near Skamokawa, Wash	1949-70	78	148	186	214	243
14248200	Jim Crow Creek Near Grays River, Wash	1965-79	560	918	1,110	1,250	1,400
14249000	Grays River Above S Fk Near Grays River, Wash	1956-79	5,370	8,320	9,720	10,700	11,700
14250500	West Fork Grays River Near Grays River, Wash	1949-69	2,420	3,820	4,590	5,180	5,800



USGS Regression Equation Region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National Oceanic and Atmospheric Administration
National Weather Service, Office of Hydrology
Prepared for U.S. Department of Agriculture,
Soil Conservation Service, Engineering Division
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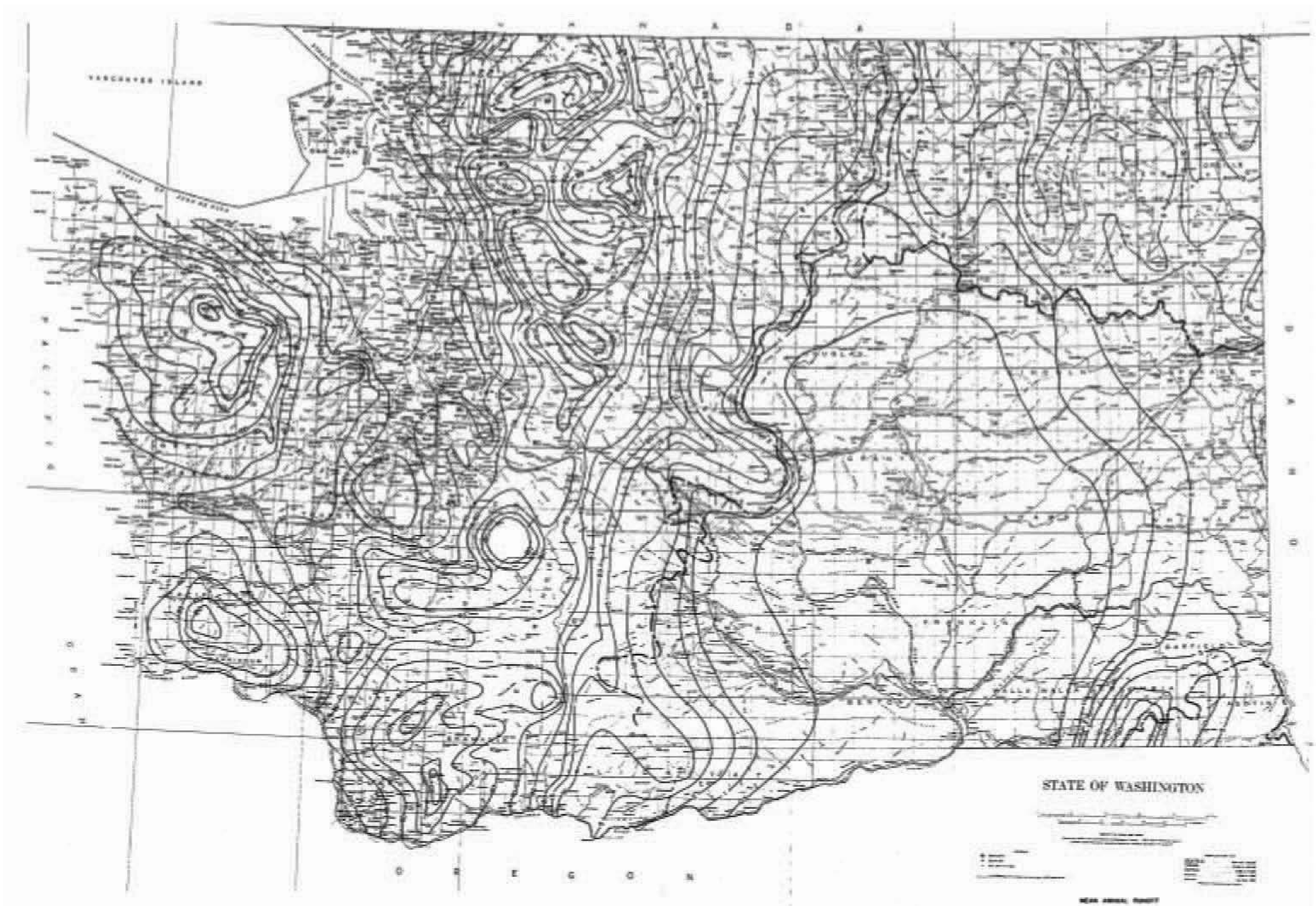
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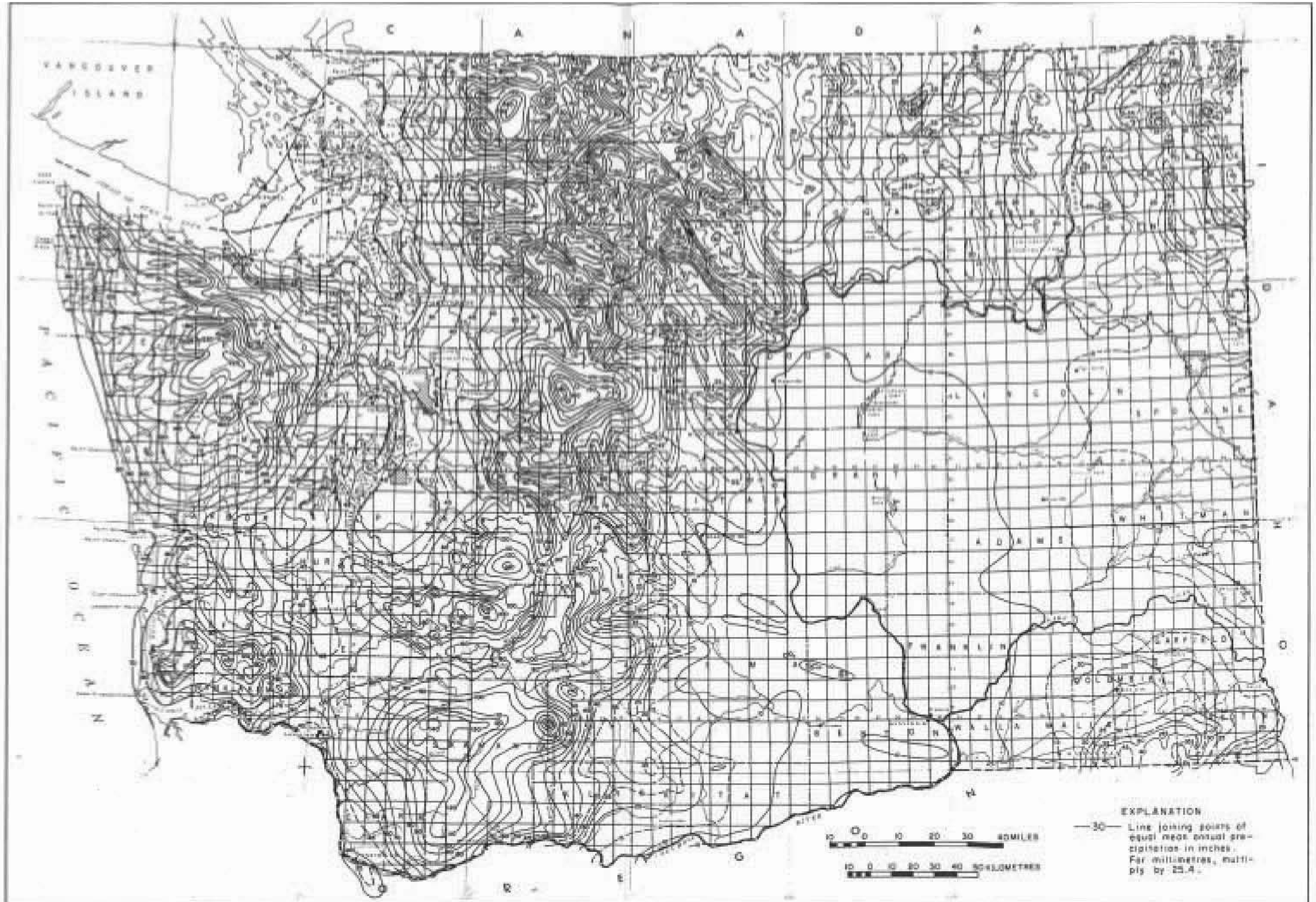


Mean annual precipitation in Washington, 1930-57. From U.S. Weather Bureau (1965).

3-1 Overview

A culvert is a closed conduit under a roadway or embankment used to maintain flow from a natural channel or drainage ditch. A culvert should convey flow without causing damaging backwater, excessive flow constriction, or excessive outlet velocities.

In addition to determining the design flows and corresponding hydraulic performance of a particular culvert, other factors can affect the ultimate design of a culvert and should be taken into consideration. These factors can include the economy of alternative pipe materials and sizes, horizontal and vertical alignment, environmental concerns, and necessary culvert end treatments.

In some situations, the hydraulic capacity may not be the only consideration for determining the size of a culvert opening. Fish passage requirements often dictate a different type of crossing than would normally be used for hydraulic capacity. Wetland preservation may require upsizing a culvert or replacement of a culvert with a bridge. Excessive debris potential may also require an increase in culvert size. In these cases, the designer should seek input from the proper authorities and document this input in the Hydraulic Report in order to justify the larger design.

3-1.1 Metric Units and English Units

When this manual was revised in 1997, WSDOT was in the process of converting to metric units. The 1997 revision included dual units throughout this chapter (and manual) except on charts and graphs. A supplement to this manual was planned that would include Metric charts and graphs, however WSDOT converted back to English units before the supplement was completed. Dual units have been left in this manual to accommodate any redesigns on metric projects. In the event a design requires metric units, it is recommended that the designer complete the form in English units and convert the discharges, controlling HW elevation, and velocity to metric units. All equations related to the charts and graphs are shown in English units only. Elsewhere in the chapter, dual units are provided.

3-2 Culvert Design Documentation

3-2.1 Common Culvert Shapes and Terminology

3-2.2 Hydraulic Reports

Culverts 1200 mm (48 inches) or less in diameter or span will be included as part of a Type B Hydraulic Report and will be reviewed by the Region Hydraulics Office/Contact as outlined in Chapter 1. The designer shall collect field data and perform an engineering analysis as described in Sections 3-2.3 and 3-2.4. Culverts in this size range should be referred to on the contract plan sheets as "Schedule ____ Culv. Pipe ____ mm (in.) Diam.". The designer is responsible for listing all acceptable pipe alternates based on site conditions. The decision regarding which type of pipe material to be installed at a location will be left to the contractor. See Chapter 8 for a discussion on schedule pipe and acceptable alternates.

Culverts larger than 1200 mm (48 inches) in diameter or span will be included as part of a Type A Hydraulic Report and will be reviewed by both the Regional Hydraulics Office/Contact and the Headquarters (HQ) Hydraulics Branch as outlined in Chapter 1. The designer shall collect field data and perform an engineering analysis as described in Sections 3-2.3 and 3-2.4.

If it is determined that a bottomless arch or three-sided box structure is required at a location, the HQ Hydraulics Branch is available to provide assistance in the design. The level of assistance provided by the HQ Hydraulics Branch can range from full hydraulic and structural design to review of the completed design. If a project office requests the HQ Hydraulics Branch to complete a design, the project office shall submit field data as described in Section 3-2.3. The engineering analysis and footing structural design will be completed by the HQ Hydraulics Branch, generally within four to six weeks after receiving field data. Once completed, the design will be returned to the project office and included as part of the Type A Hydraulic Report.

In addition to standard culvert design, the HQ Hydraulics Branch is also available to provide assistance in the design of any unique culvert installation. The requirements for these structures will vary, and it is recommended that the HQ Hydraulics Branch be contacted early in the design phase to determine what information will be necessary to complete the engineering analysis.

3-2.3 Required Field Data

Information and field data required to complete an engineering analysis includes:

1. Topographic map showing contours and the outline of the drainage area.
2. Description of the ground cover of the drainage area.
3. Soils investigation per Section 510.03(1) of the *Design Manual.*
4. Proposed roadway alignment in the vicinity of the culvert.
5. Proposed roadway cross-section at the culvert.
6. Corrosion zone location, pH, and resistivity of the site.
7. Fish passage requirements, if applicable.
8. Any other unique features that can affect design, such as low-lying structures that could be affected by excessive headwater or other consideration discussed in Section 3-5.

3-2.4 Engineering Analysis

The collected field data will be used to perform an engineering analysis. The intent of the engineering analysis is to insure that the designer considers a number of issues, including flow capacity requirements, foundation conditions, embankment construction, run-off conditions, soil characteristics, construction problems that may occur, estimated cost, environmental concerns, and any other factors that may be involved and pertinent to the design. Also, additional considerations that may affect a given culvert design are discussed in Section 3-5. Once completed, the engineering analysis will be included as part of the Hydraulic Report for the project and shall include:

1. Field data as described in Section 3-2.3.
2. Culvert hydraulic calculations as described in Section 3-3. Approved modeling software, such as HY-8 can also be in lieu of hand calculations. If the designers wish to use different software, HQ approval is required prior to submitting final designs.

3. Roadway stationing of the culvert location.
4. Culvert profile.
5. Culvert length and size.
6. Culvert material (for culverts larger than 1200 mm (48 inches)).
7. Headwater depths, water surface elevations (WSEL) and flow rates (Q) for the design flow event (generally the 25-year event) and the 100-year flow event. These items should also appear on the plan sheets for future record.
8. Roadway cross-section and roadway profile, demonstrating the maximum height of fill over the culvert.
9. Appropriate end treatment as described in Section 3-4.

3-3 Hydraulic Design of Culverts

A complete theoretical analysis of the hydraulics of a particular culvert installation is time-consuming and complex. Flow conditions vary from culvert to culvert and can also vary over time for any given culvert. The barrel of the culvert may flow full or partially full depending upon upstream and downstream conditions, barrel characteristics, and inlet geometry. However, under most conditions, a simplified procedure can be used to determine the type of flow control and corresponding headwater elevation that exist at a culvert during the chosen design flow.

This section includes excerpts from the Federal Highway Administration's **Hydraulic Design Series No. 5 — Hydraulic Design of Highway Culverts"** (HDS 5). The designer should refer to this manual for detailed information on the theory of culvert flow or reference an appropriate hydraulics textbook for unusual situations. The HQ Hydraulics Branch is also available to provide design guidance. The general procedure to follow when designing a culvert includes the following steps:

1. Calculate the culvert design flows (Section 3-3.1).
2. Determine the allowable headwater elevation (Section 3-3.2).
3. Determine the tailwater elevation at the design flow (Section 3-3.3).
4. Determine the type of control that exists at the design flow(s), either inlet control or outlet control (Section 3-3.4).
5. Calculate outlet velocities (Section 3-3.5).

3-3.1 Culvert Design Flows

The first step in designing a culvert is to determine the design flows to be used. The flow from the basin contributing to the culvert can be calculated using the methods described in Chapter 2. Generally, culverts will be designed to meet criteria for two flows: the 25-year event and the 100-year event. If fish passage is a requirement at a culvert location, an additional flow event must also be evaluated for the hydraulic option, the 10 percent exceedence flow (see Chapter 7). Guidelines for temporary culverts are described further below. The designer will be required to analyze each culvert at each of the design flows, insuring that the appropriate criteria are met.

For Circular Pipe, Box Culverts, and Pipe Arches

$Q_{10\%}$: If a stream has been determined to be fish bearing by either Region Environmental staff or Washington Department of Fish and Wildlife (WDFW) personnel and the hydraulic option is selected, the velocity occurring in the culvert barrel during the 10 percent exceedence flow must meet the requirements of Chapter 7.

Q25: The 25-year flow event should not exceed the allowable headwater, which is generally taken as 1.25 times the culvert diameter or rise as described in Section 3-3.2.2. Additionally, the WSEL for the 25-year event should not exceed the elevation of the base course of the roadway (else the base course could be saturated).

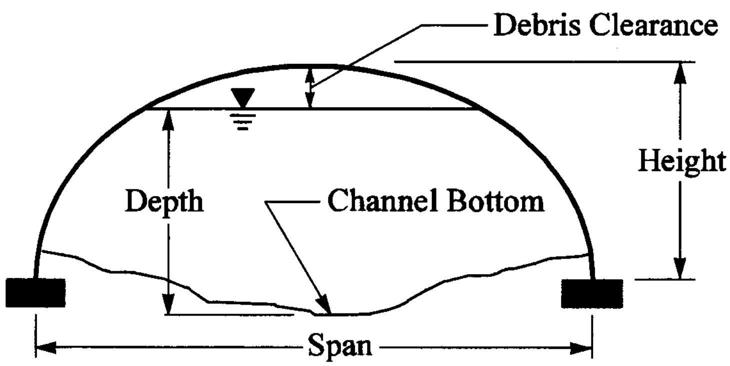
Q100: It is recommended that the culvert be sized such that there is no roadway overtopping during the 100-year flow event. See Section 3-3.2.2 for more discussion on this topic.

For Concrete or Metal Bottomless Culverts

Q10%: If a stream has been determined to be fish bearing by either Region Environmental staff or WDFW personnel <u>and</u> the hydraulic option is selected, the velocity occurring during the 10% exceedance flow through the arch must meet the requirements of Chapter 7.

Q25: 1 foot (0.3 meters) of debris clearance should be provided between the water surface and the top of the arch during the 25-year flow event, as shown in Figure 3.3.1 and discussed in Section 3-3.2.3. Additionally, the WSEL for the 25-year event should not exceed the elevation of the base course of the roadway (else the base course could be saturated).

Q100: The depth of flow during the 100-year flow event should not exceed the height of the arch as described in Section 3-3.2.3.



Typical Bottomless Culvert
Figure 3-3.1

3-3.1.1 Temporary Culverts

Temporary culverts should be sized for the 2-year storm event, unless the designer can justify a different storm event and receive HQ or Region Hydraulics approval. If the designer should decide to challenge the 2 year storm event, the designer should consider the following: the number of seasons during construction, the construction window, historical rainfall data for at least 10 years (both annually and monthly) and factor in any previous construction experience at the site.

1. Construction Seasons: If the construction season will extend beyond two seasons, the 2 year storm event should be used to size a temporary culvert. If only one season is involved, proceed to number 2.
2. Construction Window: If construction will occur during one season, the designer should evaluate at least 10 years of rainfall data for that season and then have HQ Hydraulics perform a statistical analysis to determine an appropriate peak rainfall during that season to generate a flow rate for sizing the culvert. If gage data is

available for the peak flow rate during the season of construction that should always be used first. The designer should consult the Region Hydraulics Office for further guidance. Rainfall data can be found at http://nwis.waterdata.usgs.gov/wa/nwis/dvstst.

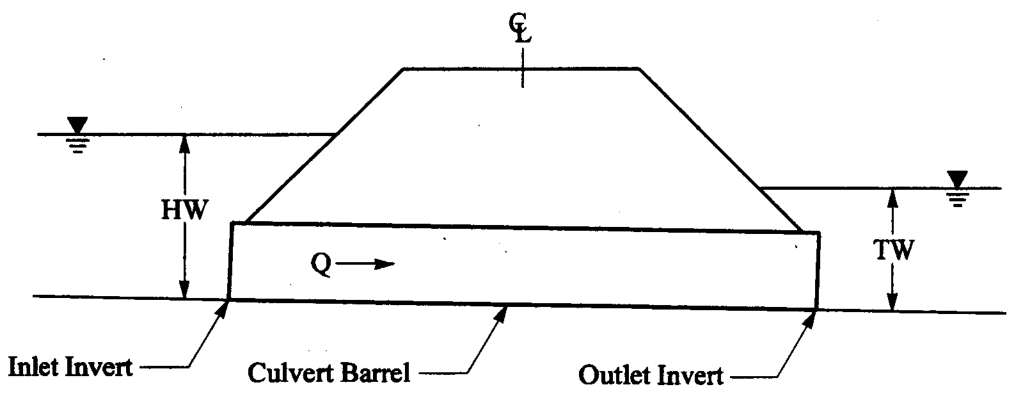
3. Previous Experience: Previous experience sizing temporary culverts at a nearby site can be the best way to size the culvert. If for example, the 2-year event yielded a 36" diameter culvert (assuming the same season), but the culvert was only 6-8 inches full, a reduction in the culvert size could be justified.

It is recommended that Region Hydraulics be involved at the beginning of this process. The designer should document the steps followed above in the Hydraulics Report.

3-3.2 Allowable Headwater

3-3.2.1 General

The depth of water that exists at the culvert entrance at a given design flow is referred to as the **headwater (HW).** Headwater depth is measured from the invert of the culvert to the water surface, as shown in Figure 3-3.2.1.



Headwater and Tailwater Diagram
Figure 3-3.2.1

Limiting the amount headwater during a design flow can be beneficial for several reasons. The potential for debris clogging becomes less as the culvert size is increased. Maintenance is virtually impossible to perform on a culvert during a flood event if the inlet is submerged more than a few feet. Also, increasing the allowable headwater can adversely impact upstream property owners by increasing flood elevations. These factors must be taken into consideration and balanced with the cost effectiveness of providing larger or smaller culvert openings.

If a culvert is to be placed in a stream that has been identified in a Federal Emergency Management Agency (FEMA) Flood Insurance Study, the floodway and floodplain requirements for that municipality may govern the allowable amount of headwater. In this situation, it is recommended that the designer contact either the Region Hydraulics Section/Contact or the HQ Hydraulics Branch for additional guidance.

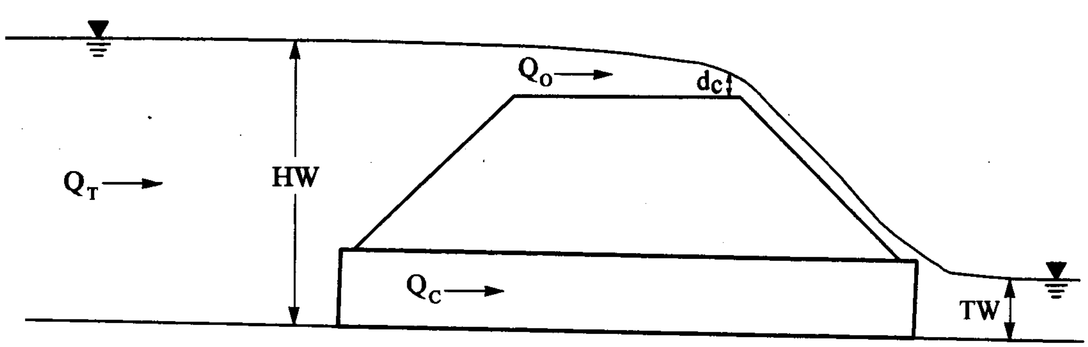
3-3.2.2 Allowable Headwater for Circular Culverts, Box Culverts, and Pipe Arches

Circular culverts, box culverts, and pipe arches should be designed such that the ratio of the headwater (HW) to diameter (D) during the 25-year flow event is less than or equal to 1.25 ($HW_i/D < 1.25$). HW_i/D ratios larger than 1.25 are permitted, provided that existing site conditions dictate or warrant a larger ratio. An example of this might be an area with high roadway fills, little stream debris, and no impacted upstream property owners. Generally, the maximum allowable HW_i/D ratios should not exceed 3 to 5. The justification for exceeding the HW_i/D ratio of 1.25 must be discussed with either the Region Hydraulics Section/Contact or the HQ Hydraulics Branch and, if approved, included as a narrative in the corresponding Hydraulics Report.

The headwater that occurs during the 100-year flow event must also be investigated. Two sets of criteria exist for the allowable headwater during the 100-year flow event, depending on the type of roadway over the culvert:

1. If the culvert is under an interstate or major state route that must be kept open during major flood events, the culvert must be designed such that the 100-year flow event can be passed without overtopping the roadway.
2. If the culvert is under a minor state route or other roadway, it is recommended that the culvert be designed such that there is no roadway overtopping during the 100-year flow event. However, there may be situations where it is more cost effective to design the roadway embankment to withstand overtopping rather than provide a structure or group of structures capable of passing the design flow. An example of this might be a low ADT roadway with minimal vertical clearance that, if closed due to overtopping, would not significantly inconvenience the primary users.

   Overtopping, of the road, will begin to occur when the headwater rises to the elevation of the roadway centerline. The flow over the roadway will be similar to flow over a broad-crested weir, as shown in Figure 3-3.2.2. A methodology is available in HDS 5 to calculate the simultaneous flows through the culvert and over the roadway. The designer must keep in mind that the downstream embankment slope must be protected from the erosive forces that will occur. This can generally be accomplished with riprap reinforcement, but the HQ Hydraulics Branch should be contacted for further design guidance. Additionally, the designer should verify the adjacent ditch does not overtop and transport runoff causing damage to either the road or private property.



Roadway Overtopping
Figure 3-3.2.2

3-3.2.3 Allowable Headwater for Bottomless Culverts

Bottomless culverts with footings should be designed such that 0.3 meters (1 foot) of debris clearance from the water surface to the culvert crown is provided during the 25-year flow event. In many instances, bottomless culverts function very similarly to bridges. They typically span the main channel and are designed to pass relatively large flows. If a large arch becomes plugged with debris, the potential for significant damage occurring to either the roadway embankment or the culvert increases. Excessive headwater at the inlet can also increase velocities through the culvert and correspondingly increase the scour potential at the footings. Sizing a bottomless culvert to meet the 0.3 meter (1 foot) criteria will alleviate many of these potential problems.

Bottomless culverts should also be designed such that the 100-year event can be passed without the headwater depth exceeding the height of the culvert. Flow depths greater than the height can cause potential scour problems near the footings.

3-3.3 Tailwater Conditions

The depth of water that exists in the channel downstream of a culvert is referred to as the **tailwater (TW)** and is shown in Figure 3-3.2.1. Tailwater is important because it can effect the depth of headwater necessary to pass a given design flow. This is especially true for culverts that are flowing in outlet control, as explained in Section 3-3.4. Generally, one of three conditions will exist downstream of the culvert and the tailwater can be determined as described below.

1. If the downstream channel is relatively undefined and depth of flow during the design event is considerably less than the culvert diameter, the tailwater can be ignored. An example of this might be a culvert discharging into a wide, flat area. In this case, the downstream channel will have little or no impact on the culvert discharge capacity or headwater.
2. If the downstream channel is reasonably uniform in cross section, slope, and roughness, the tailwater may effect the culvert discharge capacity or headwater. In this case, the tailwater can be approximated by solving for the normal depth in the channel using Manning's equation as described in Chapter 4.
3. If the tailwater in the downstream channel is established by downstream controls, other means must be used to determine the tailwater elevation. Downstream controls can include such things as natural stream constrictions, downstream obstructions, or backwater from another stream or water body. If it is determined that a downstream control exists, a method such as a backwater analysis, a study of the stage-discharge relationship of another stream into which the stream in question flows, or the securing of data on reservoir storage elevations or tidal information may be involved in determining the tailwater elevation during the design flow. If a field inspection reveals the likelihood of a downstream control, contact either the Region Hydraulics Section/Contract or the HQ Hydraulics Branch for additional guidance.

3.3.4 Flow Control

There are two basic types of flow control. A culvert flows in either inlet control or outlet control.

When a culvert is in **Inlet Control,** the inlet is controlling the amount of flow that will pass through the culvert. Nothing downstream of the culvert entrance will influence the amount of headwater required to pass the design flow.

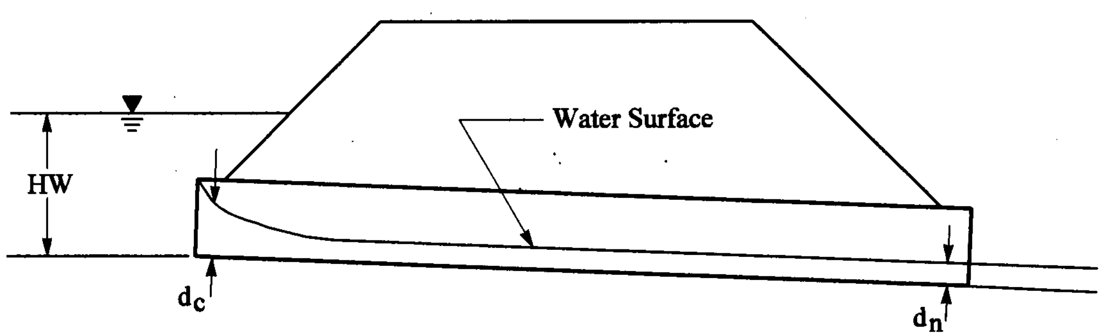
When a culvert is in **Outlet Control** , the outlet conditions or barrel are controlling the amount of flow passing through the culvert. The inlet, barrel, or tailwater characteristics, or some combination of the three, will determine the amount of headwater required to pass the design flow.

There are two different methods used to determine the headwater, one for inlet control and one for outlet control. If the culvert is flowing in inlet control, the headwater depth is calculated using inlet control equations. If the culvert is flowing in outlet control, the headwater depth is calculated using outlet control equations. Often, it is not known whether a culvert is flowing in inlet control or outlet control before a design has been completed. It is therefore necessary to calculate the headwater that will be produced for both inlet and outlet control, and then compare the results. The larger headwater will be the one that controls and that headwater will be the one that will be used in the design of the culvert. Both inlet control and outlet control will be discussed in the following sections and methods for determining the headwater for both types of control will be given.

3-3.4.1 Culverts Flowing With Inlet Control

In inlet control, the flow capacity of a culvert is controlled at the entrance by depth of headwater and the entrance geometry. The entrance geometry includes the inlet area, shape, and type of inlet edge. Changing one of these parameters, such as increasing the diameter of the culvert or using a hydraulically more efficient opening, is the only way to increase the flow capacity through the culvert for a given headwater. Changing parameters downstream of the entrance, such as modifying the culvert slope, barrel roughness, or length will not increase the flow capacity through the culvert for a given headwater.

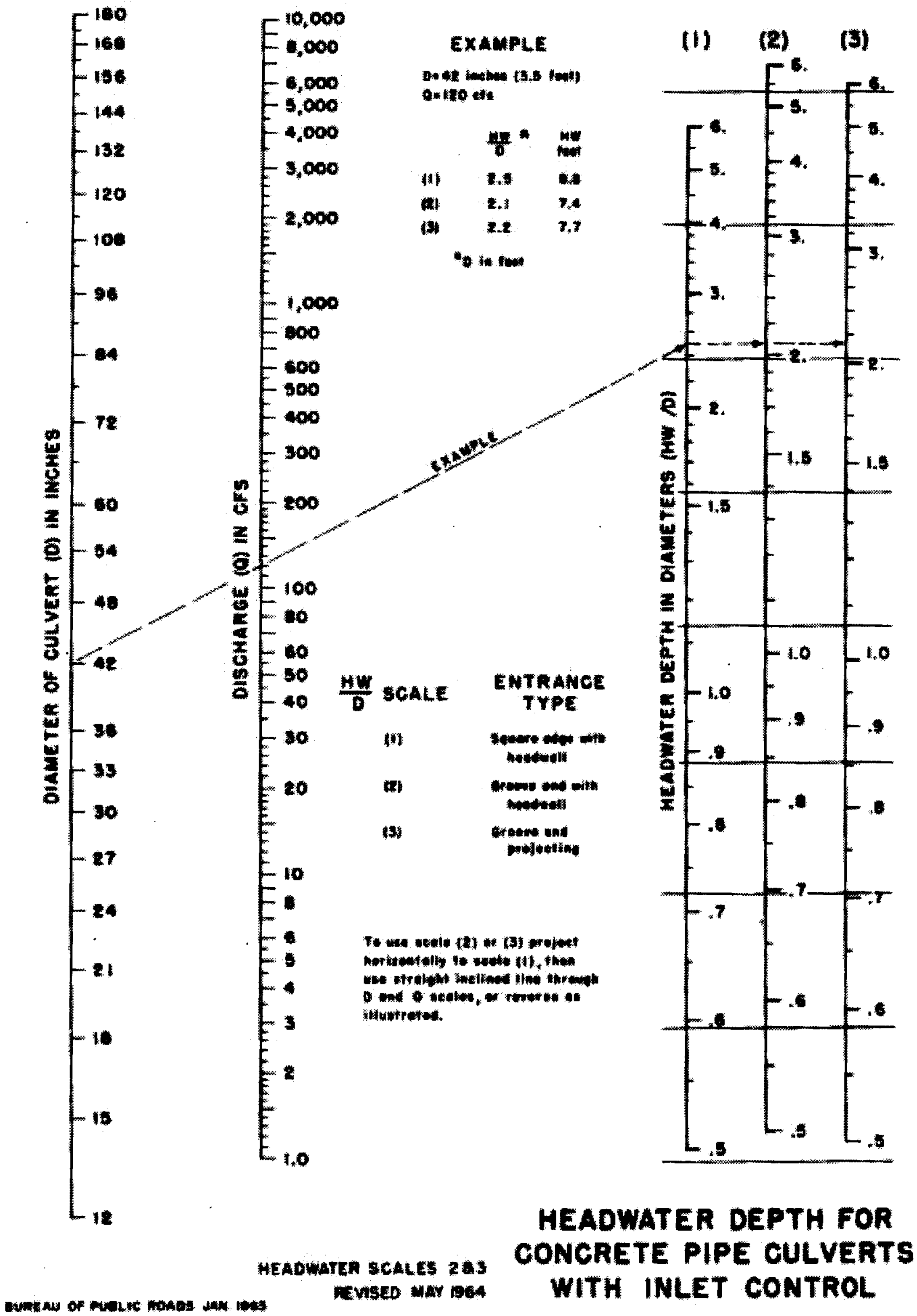
Inlet control usually occurs when culverts are placed on slopes steeper than a 1 percent grade and when there is minimal tailwater present at the outlet end. Figure 3-3.4.1 shows a typical inlet control flow profile. In the figure, the inlet end is submerged, the outlet end flows freely, and the barrel flows partly full over its length. The flow passes through critical depth (d_c) just downstream of the culvert entrance and the flow approaches normal depth (d_n) at the downstream end of the culvert.



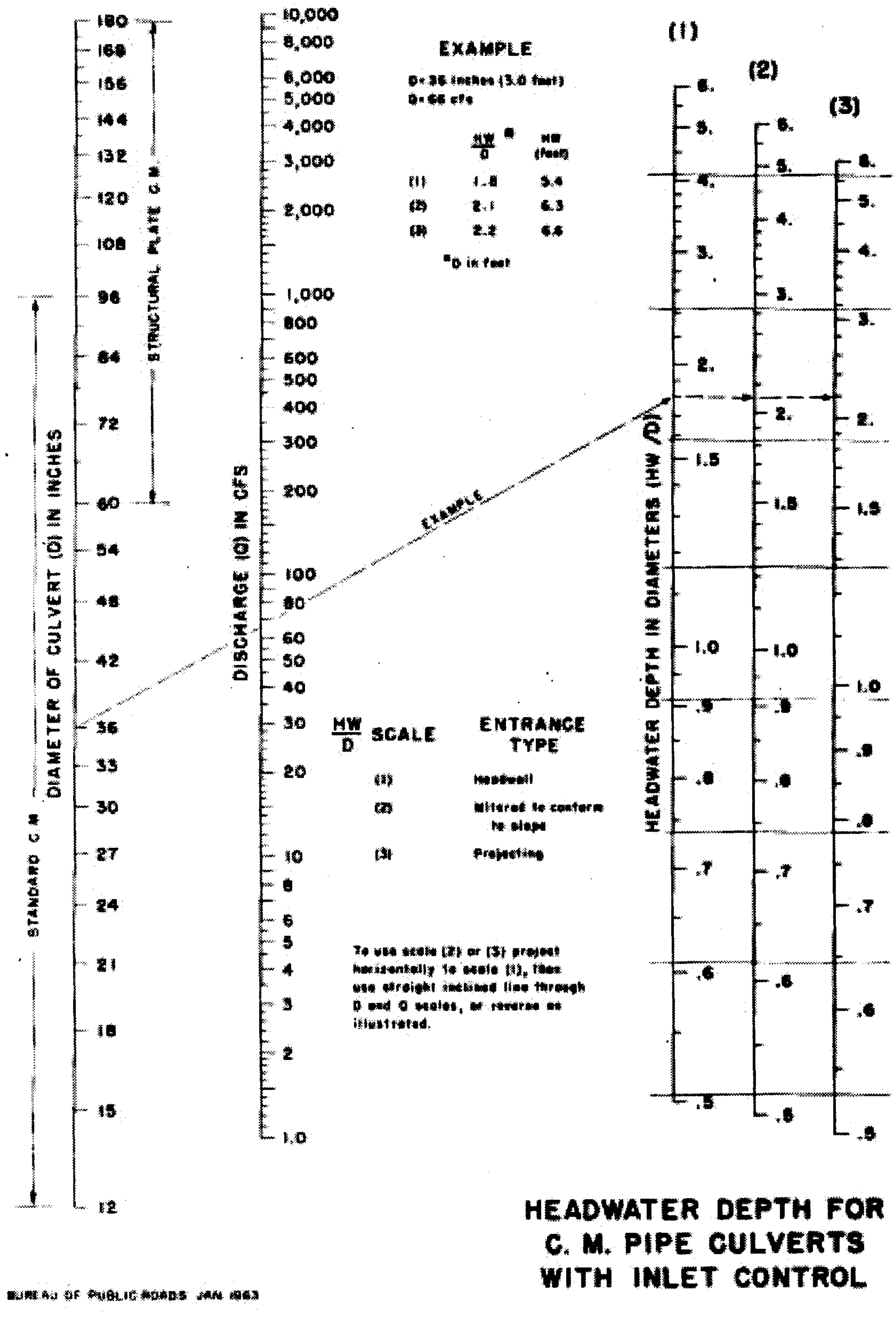
Typical Inlet Control Flow Profile
Figure 3-3.4.1

3-3.4.2 Calculating Headwater for Inlet Control

When a culvert is flowing in inlet control, two basic conditions exist. If the inlet is submerged, the inlet will operate as an orifice. If the inlet is unsubmerged, the inlet will operate as a weir. Equations have been developed for each condition and the equations demonstrate the relationship between headwater and discharge for various culvert materials, shapes, and inlet configurations. The inlet control nomographs shown Figures 3-3.4.2A-E utilize those equations and can be used to solve for the headwater.

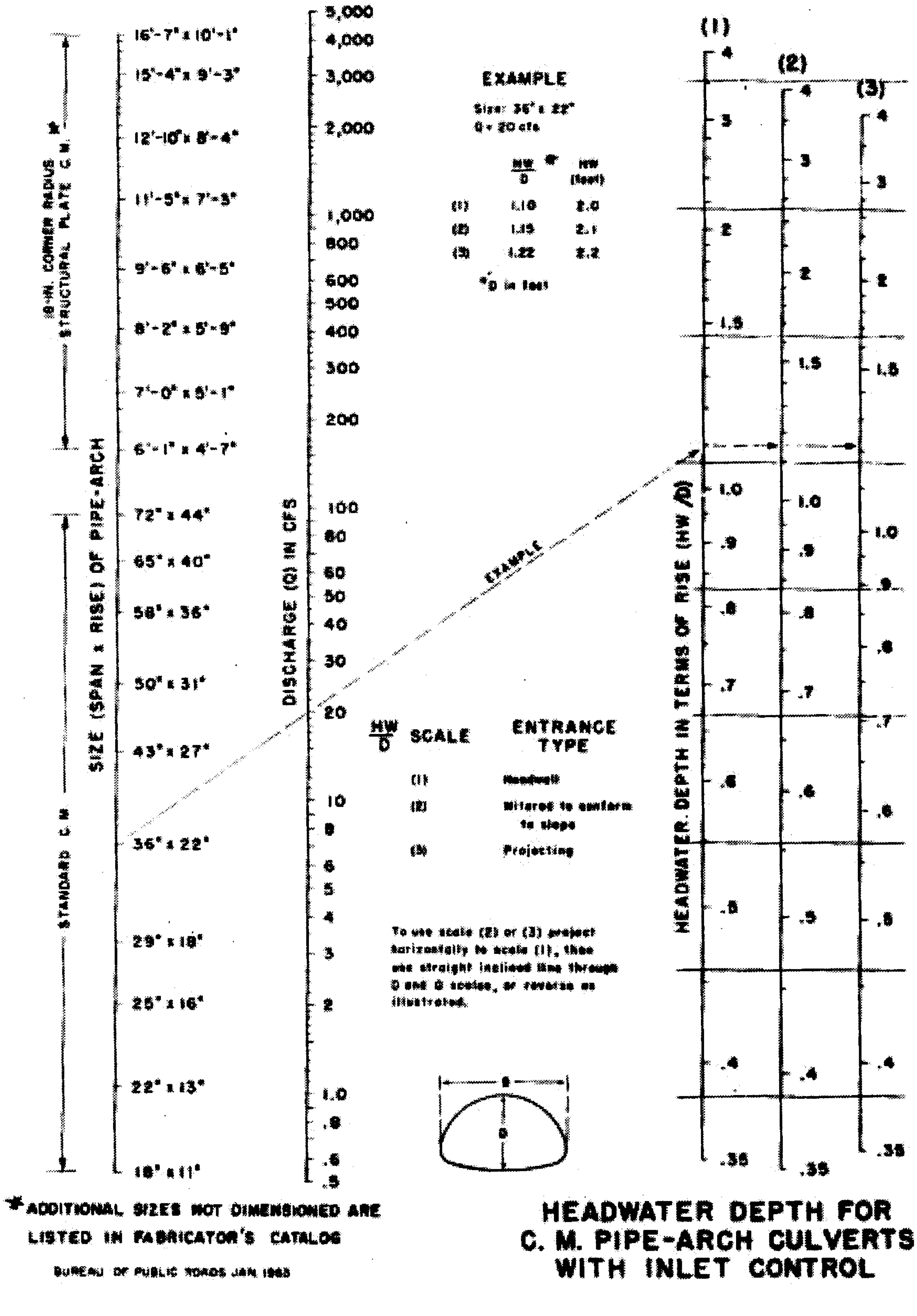


Concrete Pipe Inlet Control Nomograph
Figure 3-3.4.2A

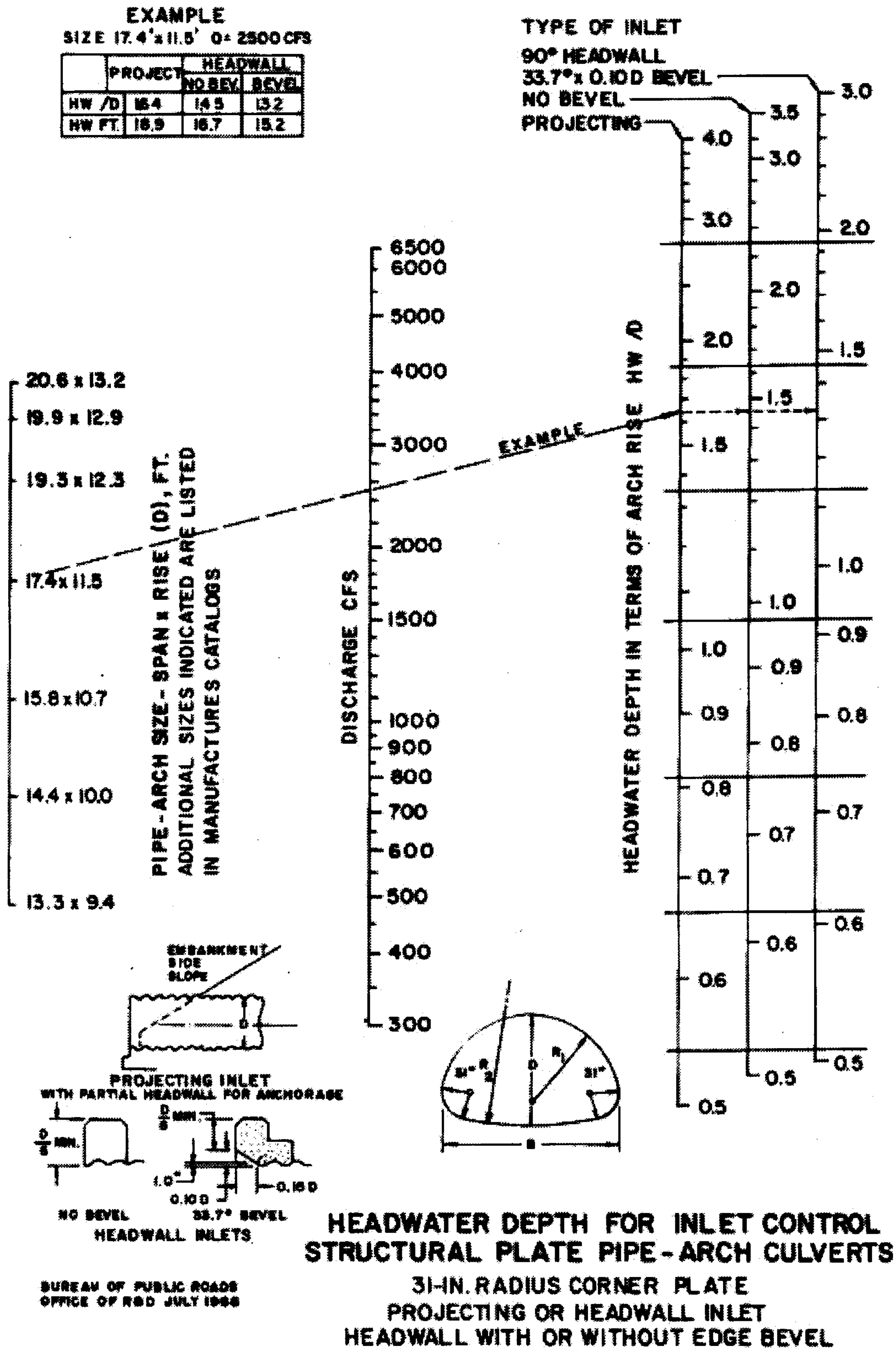


Corrugated Metal and Thermoplastic Pipe Inlet Control Nomograph

Figure 3-3.4.2B

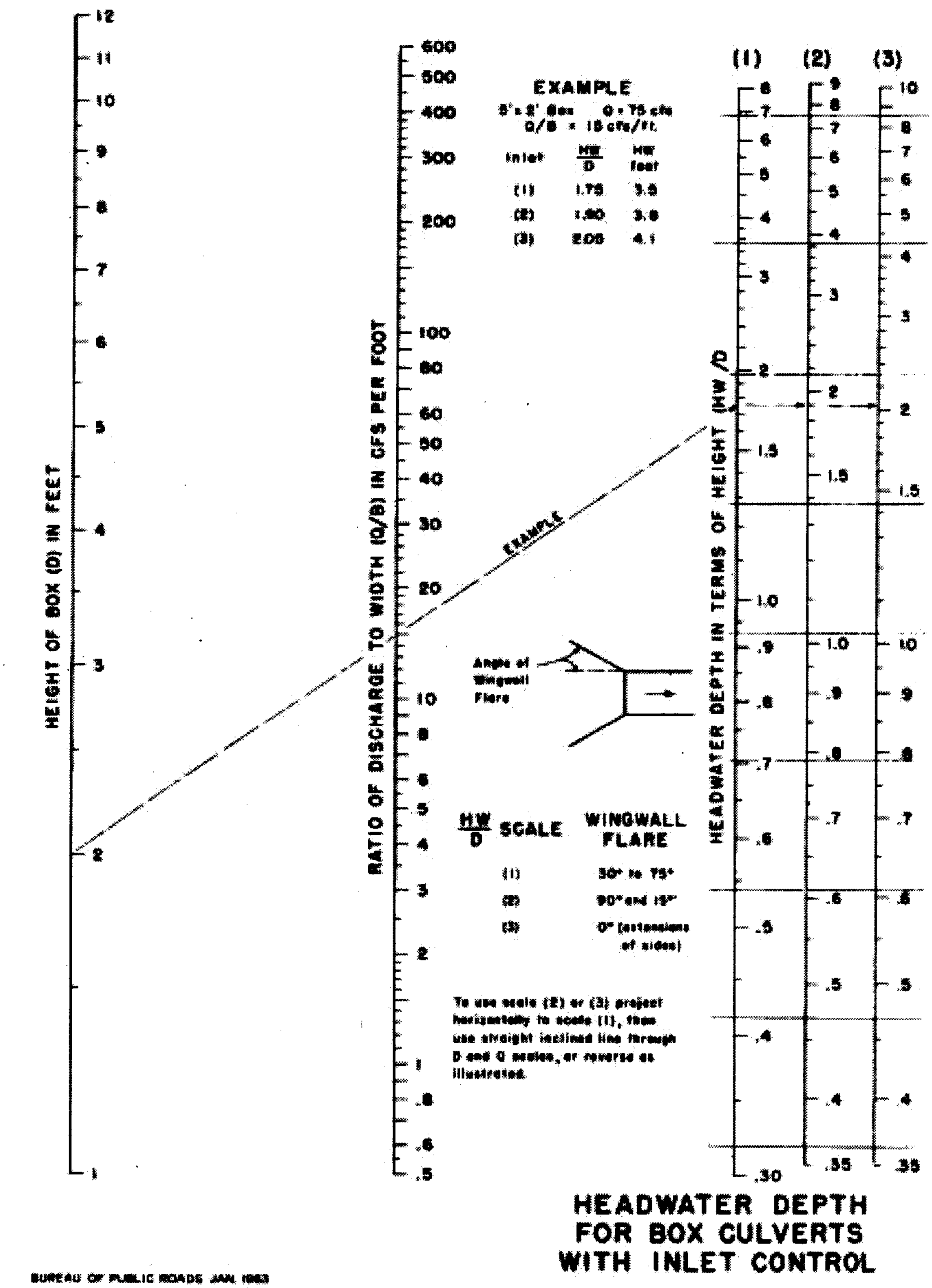


Corrugated Metal Pipe-Arch Inlet Control Nomograph
Standard Sizes and 18-Inch Corner Radius
Figure 3-3.4.2C



Corrugated Metal Pipe-Arch Inlet Control Nomograph Large Sizes

Figure 3-3.4.2D



Box Culvert Inlet Control Nomograph

Figure 3-3.4.2E

3-3.4.3 Culverts Flowing With Outlet Control

In outlet control, the flow capacity of a culvert is controlled by the inlet, barrel, or tailwater conditions, or some combination of the three. Changing any parameter, such as the culvert size, entrance configuration, slope, roughness, or tailwater condition can have a direct impact on the headwater required to pass the design flow.

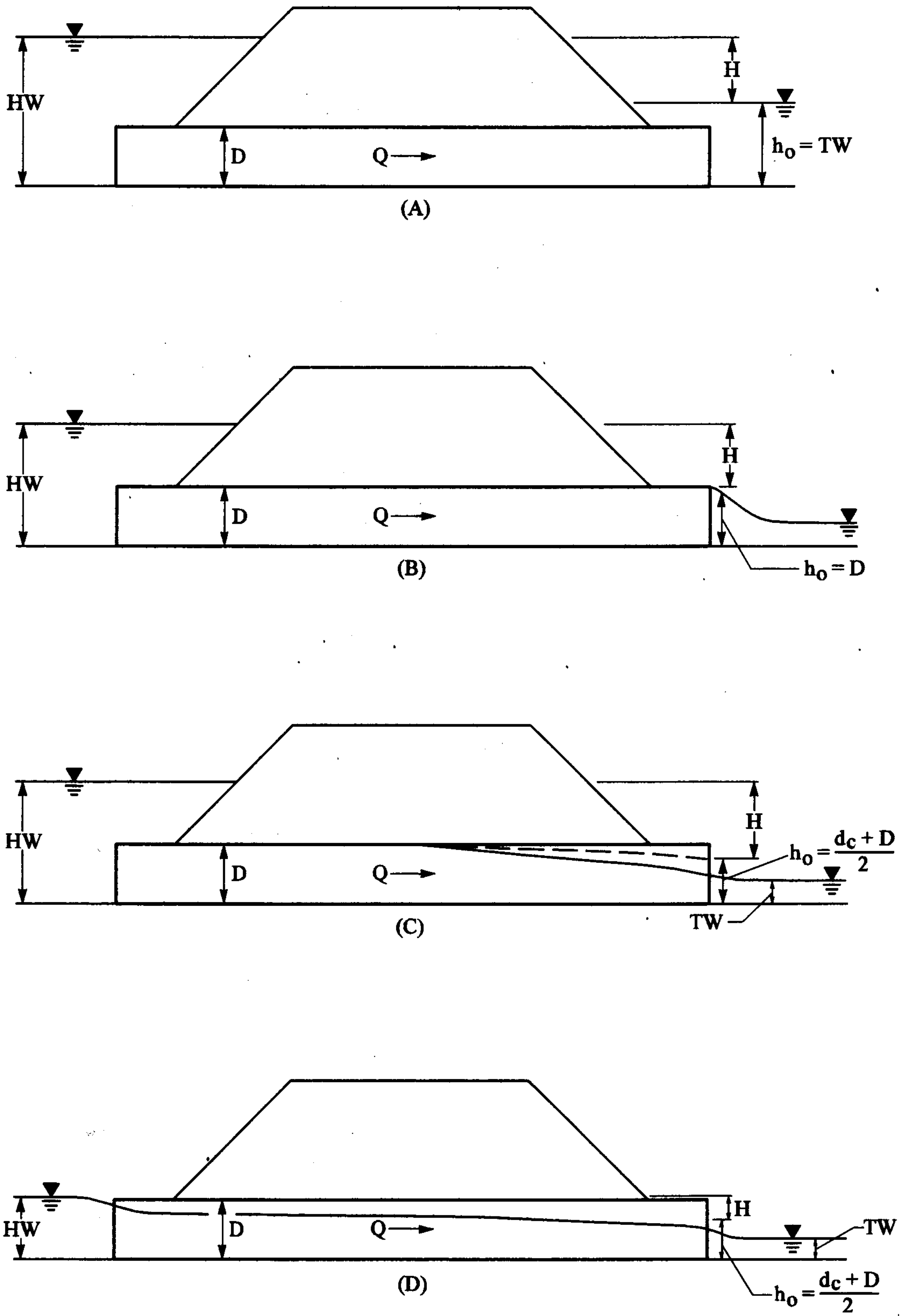
Outlet control usually occurs when a culvert is placed on a relatively flat slope, generally less than a 1 percent grade, or when the depth of tailwater is significant. Figure 3-3.4.3 demonstrates several typical outlet control flow profiles that can occur in a culvert. The method for computing the headwater for each of the profiles is the same and is described in Section 3-3.4.4. However, the method used to calculate outlet velocities for outlet control can vary as described in Section 3-3.5.2. Figure 3-3.4.3 can be useful for visually representing some of the concepts discussed in that section.

Figure 3-3.4.3(A) shows a full flow condition, with both the inlet and outlet submerged. The culvert barrel is in pressure flow throughout the entire length. This condition is often assumed in calculations but seldom actually exists.

Figure 3-3.4.3(B) shows the entrance submerged to such a degree that the culvert flows full throughout the entire length. However, the exit is unsubmerged by tailwater.This is a rare condition because it requires an extremely high headwater to maintain full barrel flow with no tailwater. The outlet velocities are unusually high under this condition.

Figure 3-3.4.3(C) is more typical. The culvert entrance is submerged by the headwater and the outlet flows freely with a low tailwater. For this condition the barrel flows partly full over at least part of its length and the flow passes through critical depth just upstream of the outlet.

Figure 3-3.4.3(D) is also typical, with neither the inlet nor the outlet end of the culvert submerged. The barrel flows partly full over its entire length. The procedure described in Section 3-3.4.4 for calculating headwater for outlet control flow does not give an exact solution in this case. However, the procedure is considered accurate when the headwater is .75D and greater, where D is the height or rise of the culvert barrel.

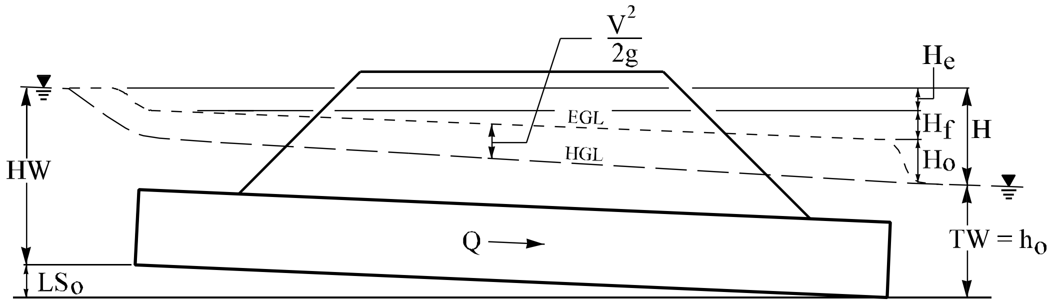


Outlet Control Flow Profiles
Figure 3-3.4.3

3-3.4.4 Calculating Headwater For Outlet Control

Outlet control headwater (HW) cannot be solved for directly. Rather, HW can be found by utilizing the relationship shown in Equation (1) and Figure 3-3.4.4A.

$$HW = H + h_o + LS_o \qquad (1)$$



Outlet Control Flow Relationships

Figure 3-3.4.4A

Where: HW = Headwater (ft)

- H = Total head loss through the culvert, including entrance, barrel, and exit losses
- h_o = Approximation of the hydraulic grade line at the outlet of the culvert (ft)
- LS_o = Product of the culvert length multiplied by the culvert slope (ft)
- EGL = Energy Grade Line. The EGL represents the total energy at any point along the culvert barrel.
- HGL = Hydraulic Grade Line. Outside of the culvert, the HGL is equal to the water surface elevation. Inside the culvert, the HGL is the depth to which water would rise in vertical tubes connected to the sides of the culvert barrel.

H, h_o, and LS_o can be calculated as described below, then used in conjunction with Equation 1 to determine HW.

H: H is the total head loss through the culvert, generally expressed in units of feet. It is made up of three major parts: an entrance loss H_e, a friction loss through the barrel H_f, and an exit loss at the outlet H_o. Expressed in equation form, the total head loss is:

$$H = H_e + H_f + H_o \qquad (2)$$

Each of the losses are a function of the velocity head in the barrel. The velocity head is the kinetic energy of the water in the culvert barrel. The velocity head is equal to $V^2/2g$, where V is the mean velocity in the culvert barrel. The mean velocity is found by dividing the discharge by the cross-sectional area of the flow.

The entrance loss H_e is found by multiplying the velocity head by an entrance loss coefficient k_e and is shown by Equation (3). The coefficient k_e for various types of culvert entrances can be found in Figure 3-3.4.5H.

$$H_e = k_e \frac{V^2}{2g} \qquad (3)$$

The friction loss H_f is the energy required to overcome the roughness of the culvert barrel. It is found by multiplying the velocity head by an expression of Manning's equation and is given by Equation (4).

$$H_f = \left[\frac{29n^2L}{R^{1.33}}\right]\frac{V^2}{2g} \qquad (4)$$

where:

- n = Manning's roughness coefficient
- L = Length of culvert barrel (ft)
- V = Man velocity of flow in culvert barrel (ft/s)
- R = Hydraulic radius (ft) (R=D/4 for full flow pipe, see Section 4-3)

The exit loss at the outlet H_o occurs when flow suddenly expands after leaving the culvert. It is found by multiplying the velocity head by an exit loss coefficient, generally taken as 1.0, and is given by Equation (5).

$$H_o = 1.0\,\frac{V^2}{2g} \qquad (5)$$

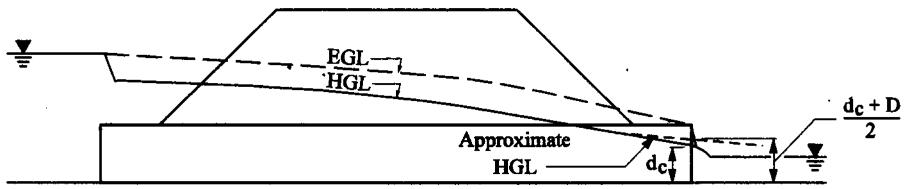
Combining Equations (3), (4), and (5) and substituting back into (2), the total head loss H can be expressed as:

$$H = \left[1 + k_e + \frac{29n^2L}{R^{1.33}}\right]\frac{V^2}{2g} \qquad (6)$$

The outlet control nomographs shown in Section 3-3.4.5 provide graphical solutions to Equation (6) and should be utilized to solve for H.

h_o: h_o is an approximation of the hydraulic grade line at the outlet of the culvert and is equal to the tailwater or $(d_c + D)/2$, whichever is greater. The term $(d_c + D)/2$ represents an approximation of the hydraulic grade line at the outlet of the culvert, where d_c is equal to the critical depth at the outlet of the culvert and D is the culvert diameter or rise. When free surface flow occurs in a culvert operating in outlet control, the most accurate method for determining the HW elevation is to perform a backwater analysis through the culvert. This, however, can be a tedious and time-consuming process. Making the assumption that $(d_c + D)/2$ represents the hydraulic grade line simplifies the design procedure. The approximate method will produce reasonably accurate results when the headwater is 0.75 D and greater, where D is the culvert diameter or rise. In situations where the headwater is less than 0.75 D, the culvert should be designed using a computer software program, as discussed in Section 3-3.7. Most programs will perform a backwater analysis through the culvert and arrive at a more accurate solution for the headwater elevation than the approximate method.

As shown in Figure 3-3.4.4B, $(d_c + D)/2$ does not represent the actual water surface elevation at the outlet of the culvert and therefore should not be used for determining the corresponding outlet velocity. The method for determining the outlet velocity is discussed in Section 3-3.5.2



Hydraulic Grade Line Approximation

Figure 3-3.4.4B

LS_o: LS_o is the culvert length (L) multiplied by the culvert slope (S_o), expressed in feet.

3-3.4.5 Outlet Control Nomographs

The outlet control nomographs presented in this section allow the designer to calculate H, the total head loss through the culvert, as discussed in Section 3-3.4.4. The nomographs should be used in conjunction with Figure 3-3.6, Culvert Hydraulic Calculations Form.

Figure 3-3.4.5A shows a sample outlet control nomograph. The following set of instructions will apply to all of the outlet control nomographs in this section. To determine H for a given culvert and discharge:

Step 1: Locate the appropriate nomograph for type of culvert selected.

Step 2: Find the Manning's n value for the culvert from Appendix 4-1. If the Manning's n value given in the nomograph is different than the Manning's n for the culvert, adjust the culvert length using the formula:

$$L_1 = L\left[\frac{n_1}{n}\right]$$

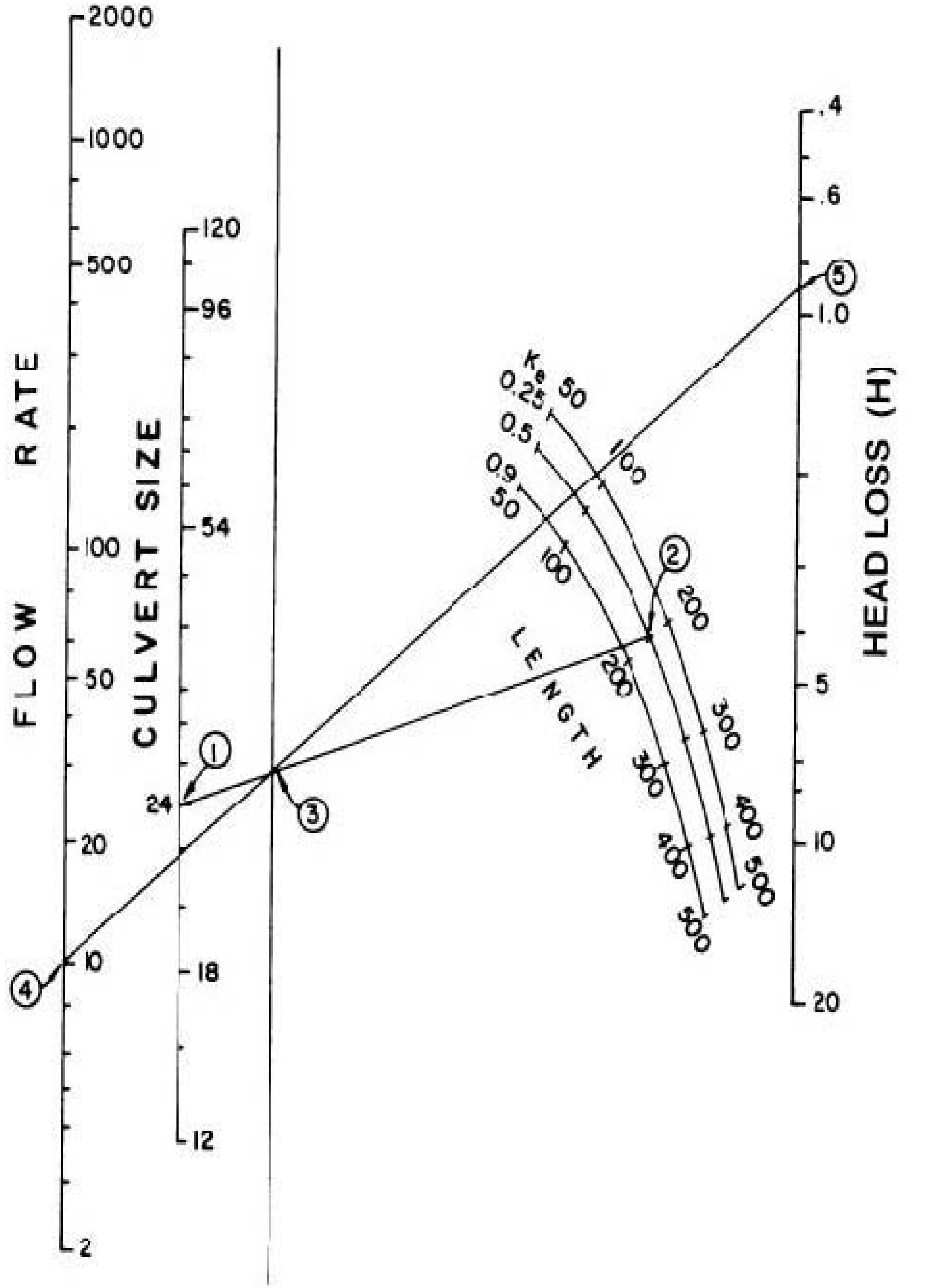
where:
- L_1 = Adjusted culvert length (ft)
- L = Actual culvert length (ft)
- n_1 = Actual Manning's n value of the culvert
- n = Manning's n value from the nomograph

Step 3: Using a straightedge, connect the culvert size (point 1) with the culvert length on the appropriate k_e curve (point 2). This will define a point on the turning line (point 3). If a k_e curve is not shown for the selected k_e, interpolate between the two bounding k_e curves. Appropriate k_e factors are shown in Figure 3-3.4.5H.

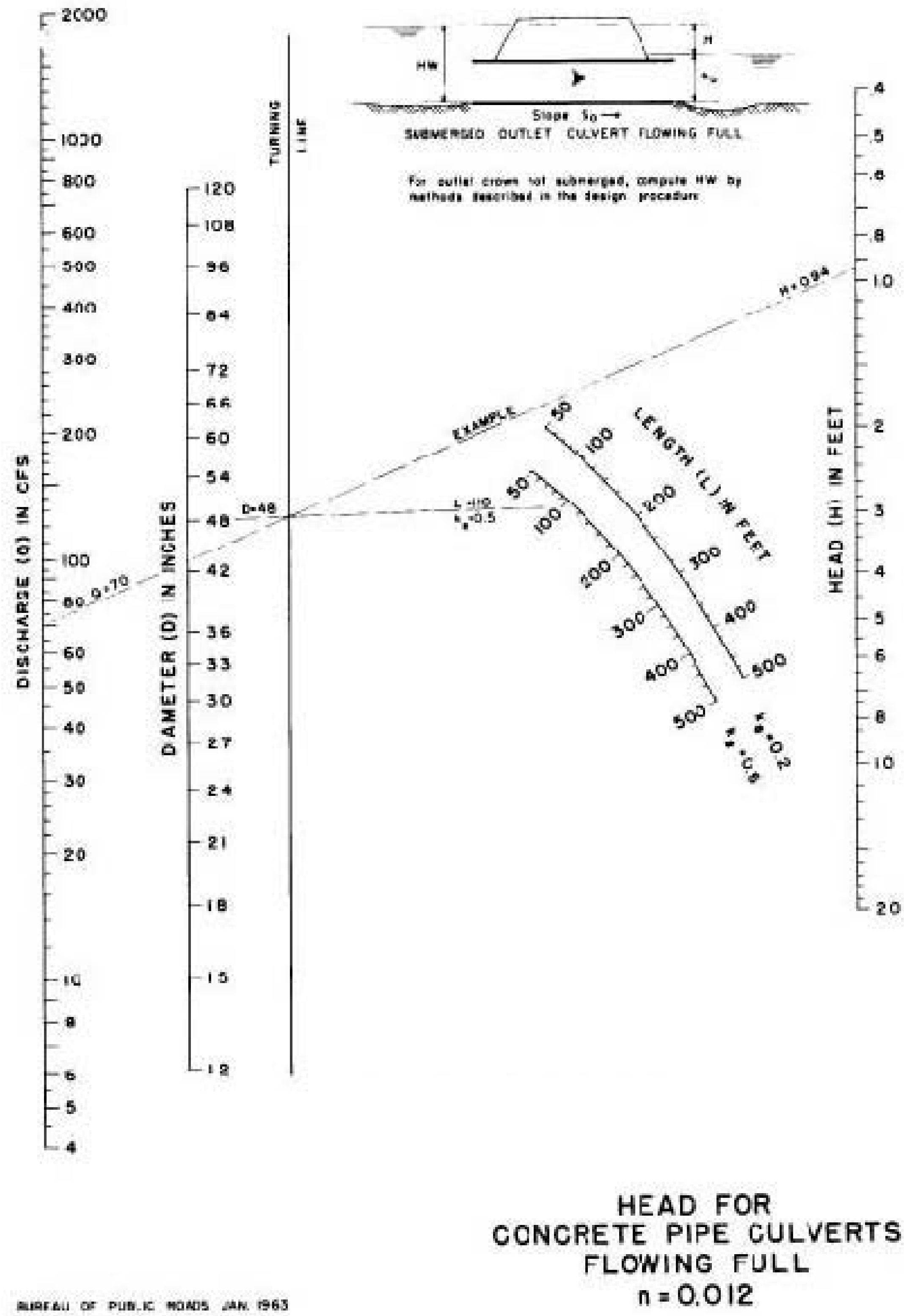
Step 4: Again using a straightedge, extend a line from the discharge (point 4) through the point on the turning line (point 3) to the head loss H scale (point 5). Read H.

Note: Careful alignment of the straightedge is necessary to obtain accurate results from the nomographs.

Figure 3-3.4.5G is the outlet control nomograph to be used for square box culverts. The nomograph can also be used for rectangular box culverts by calculating the cross-sectional area of the rectangular box and using that area as point 1 described in Step 3 above.

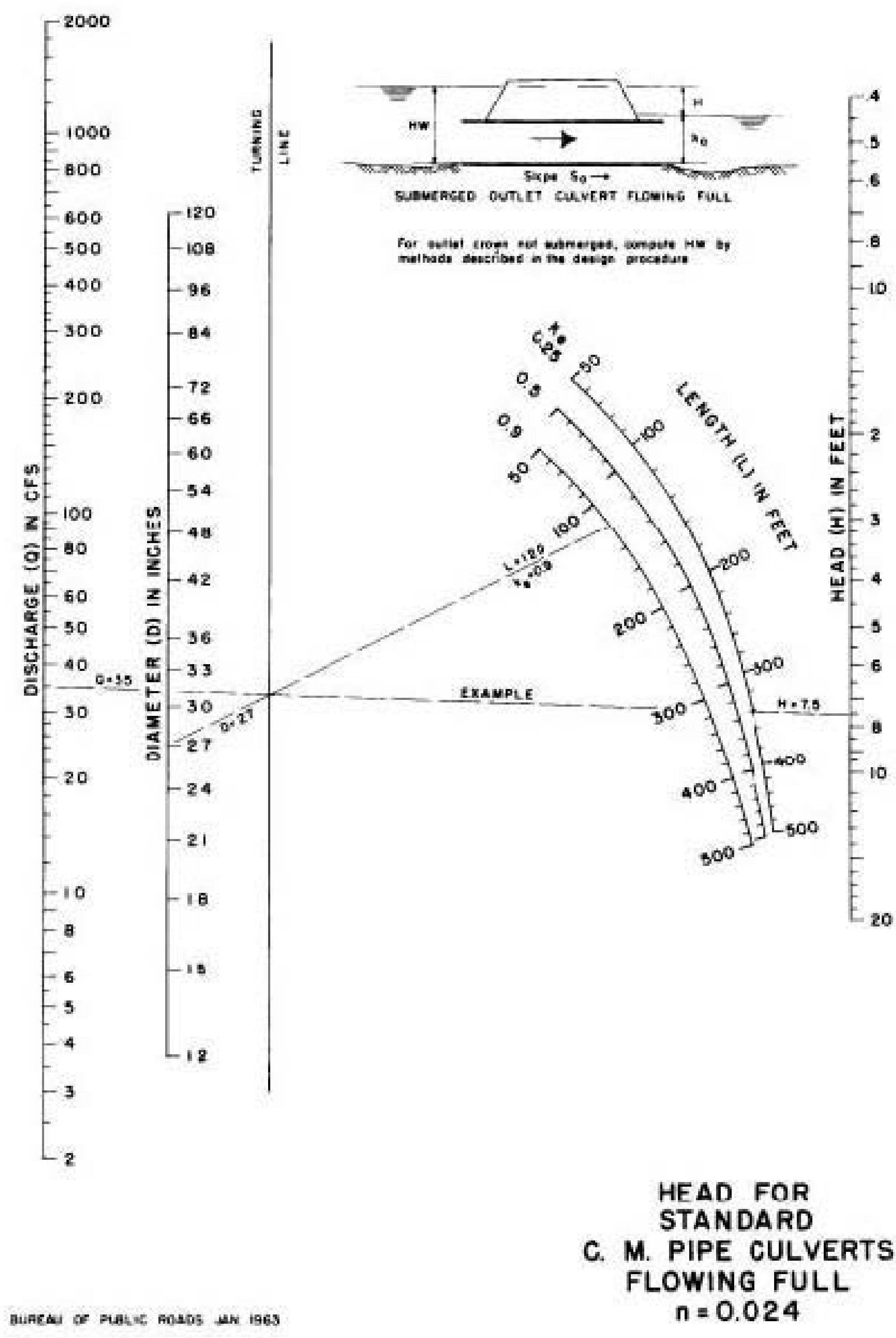


Sample Outlet Control Nomograph
Figure 3-3.4.5A



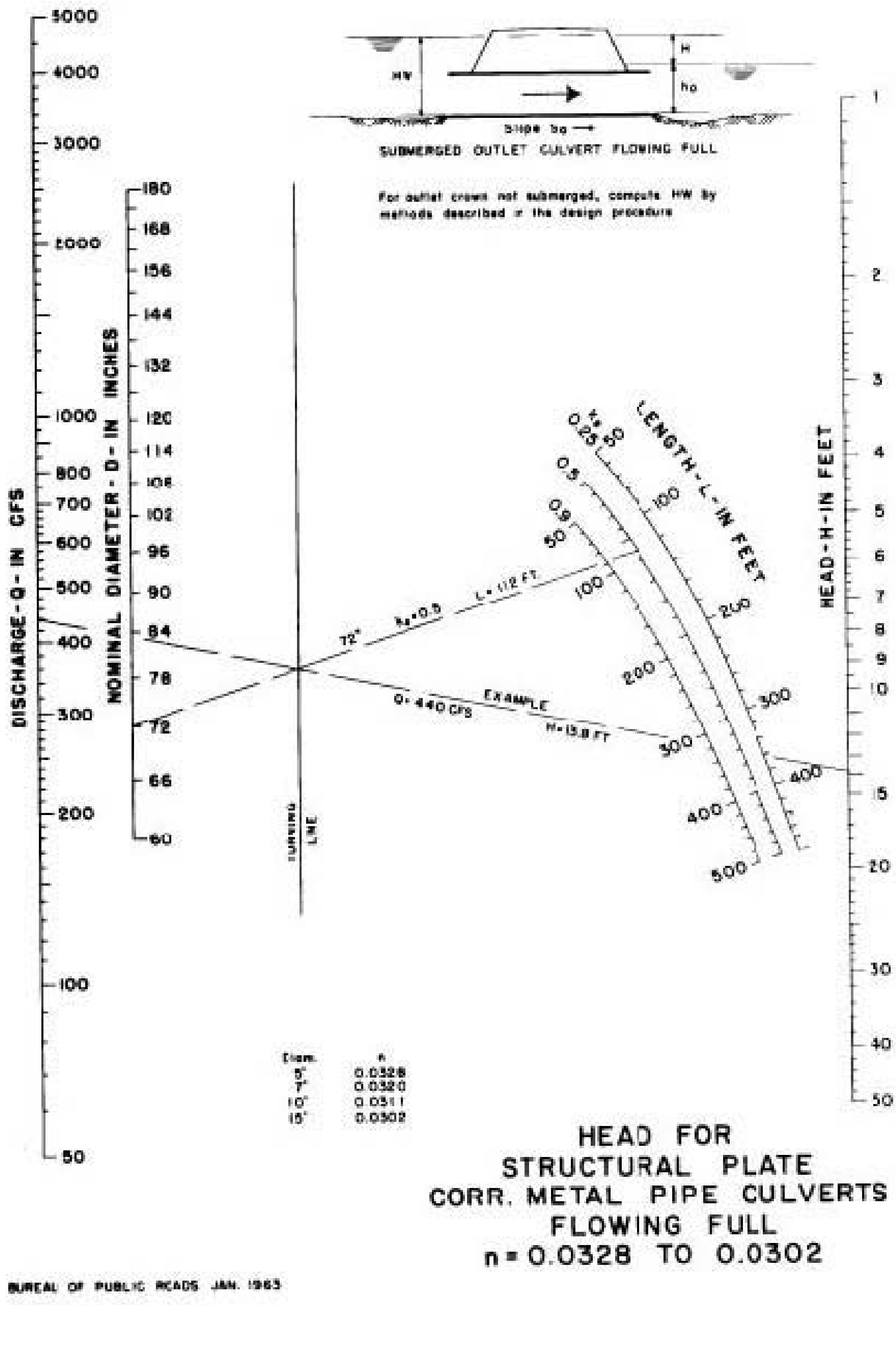
Concrete and Thermoplastic Pipe Outlet Control Nomograph

Figure 3-3.4.5B



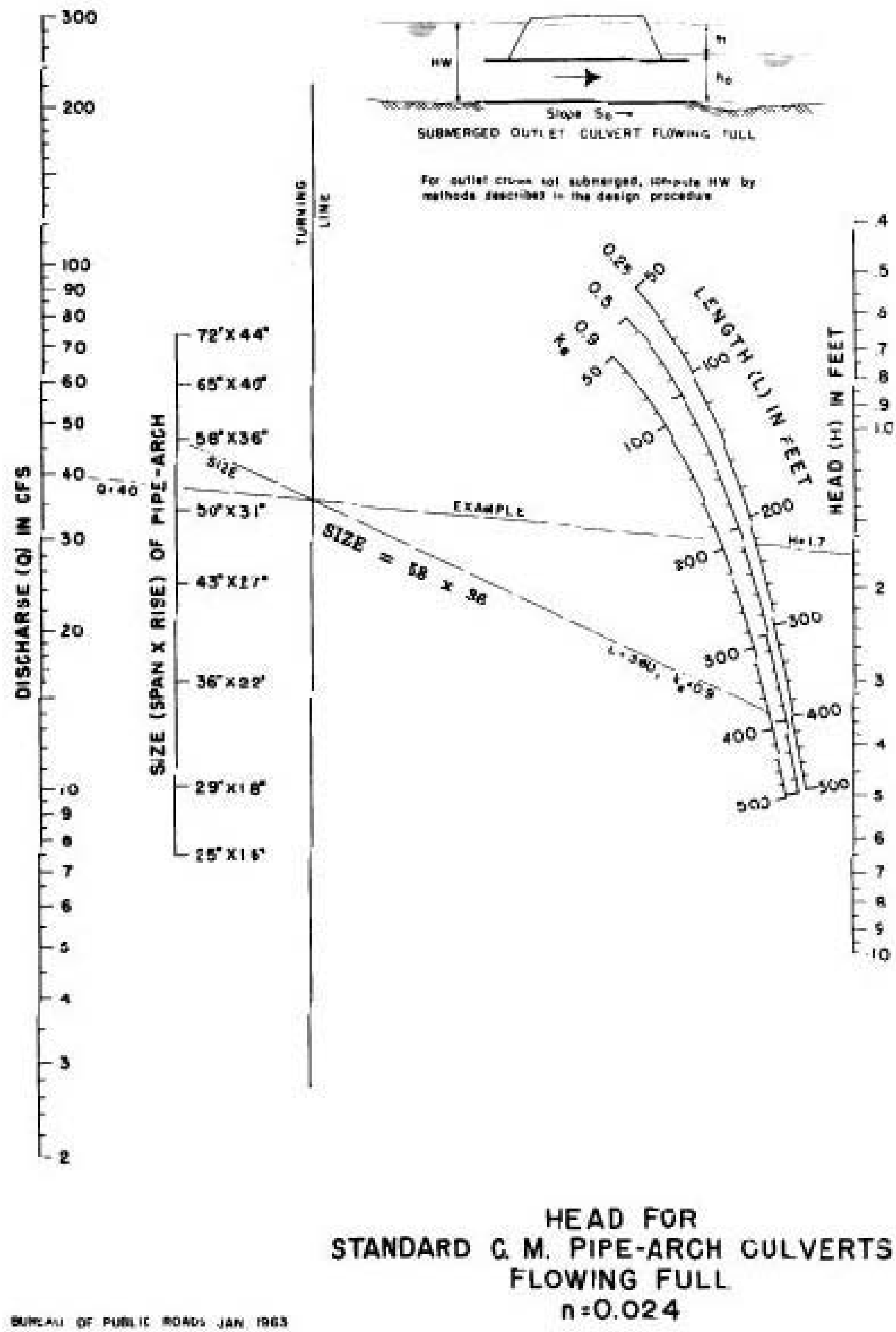
Corrugated Metal Pipe Outlet Control Nomograph

Figure 3-3.4.5C



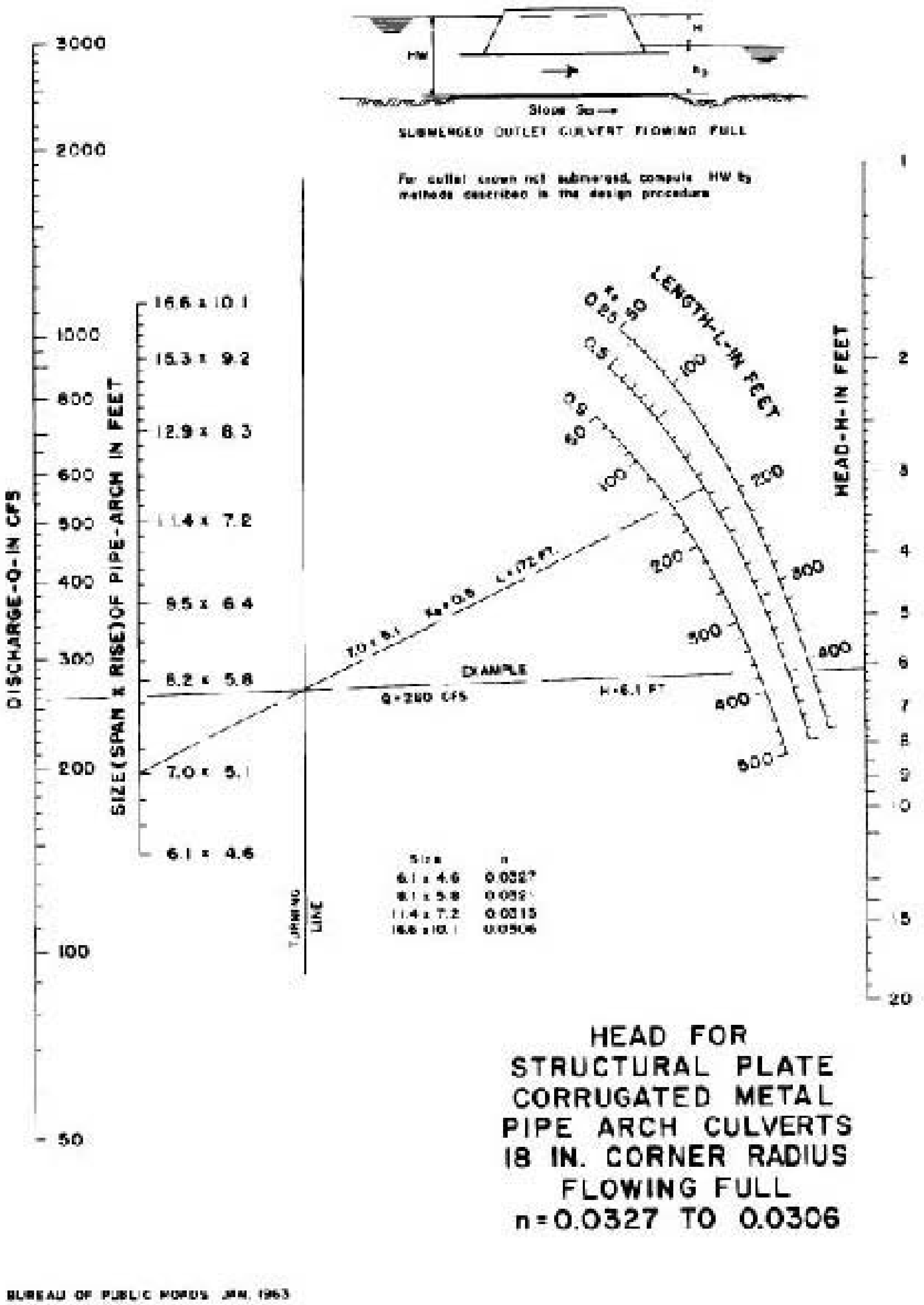
Structural Plat Corrugated Metal Pipe Outlet Control Nomograph

Figure 3-3.4.5D

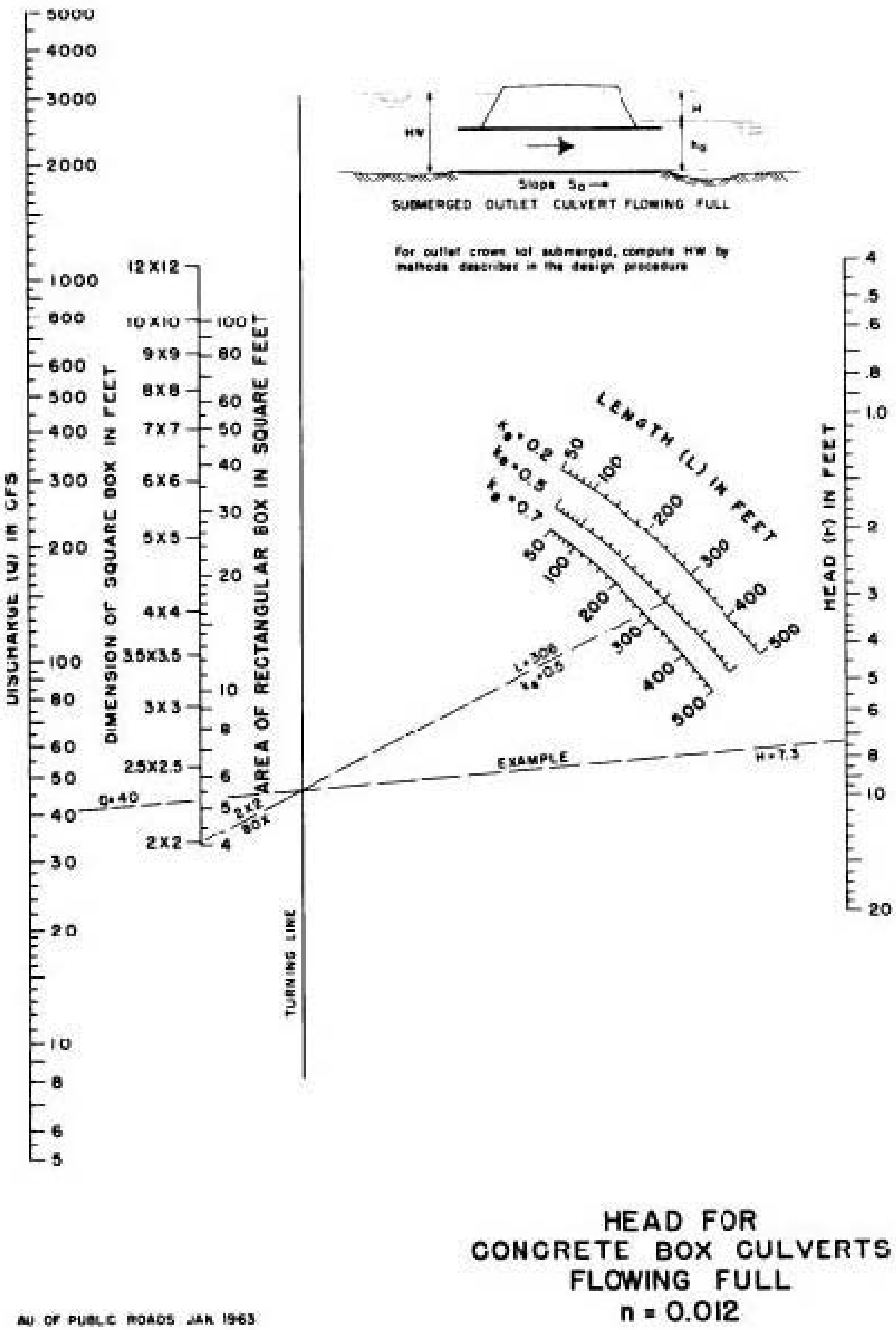


Corrugated Metal Pipe-Arch Outlet Control Nomograph

Figure 3-3.4.5E



Corrugated Metal Pipe-Arch Outlet Control Nomograph
18 Inch Corner Radius
Figure 3-3.4.5F



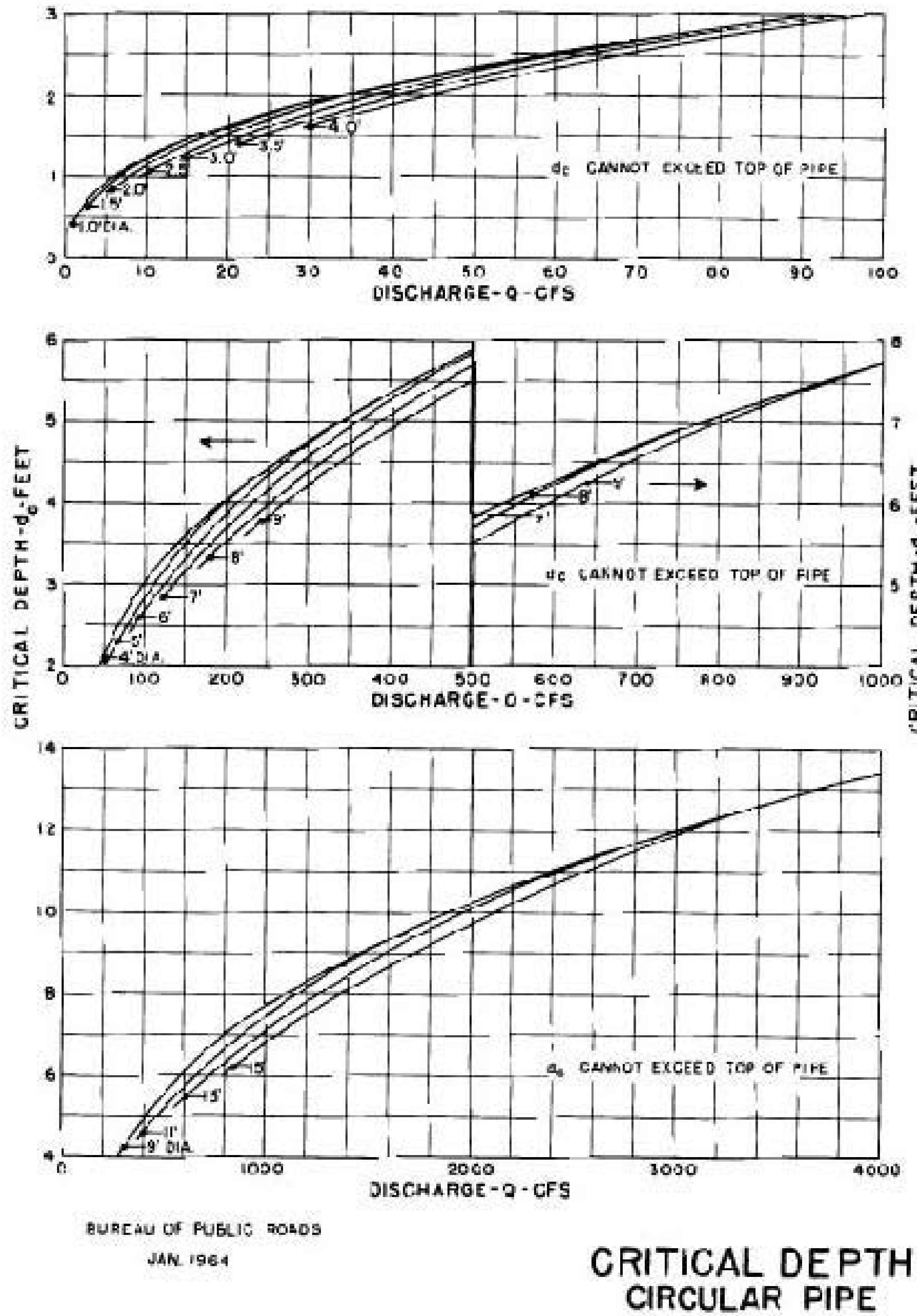
Box Culvert Outlet Control Nomograph

Figure 3-3.4.5G

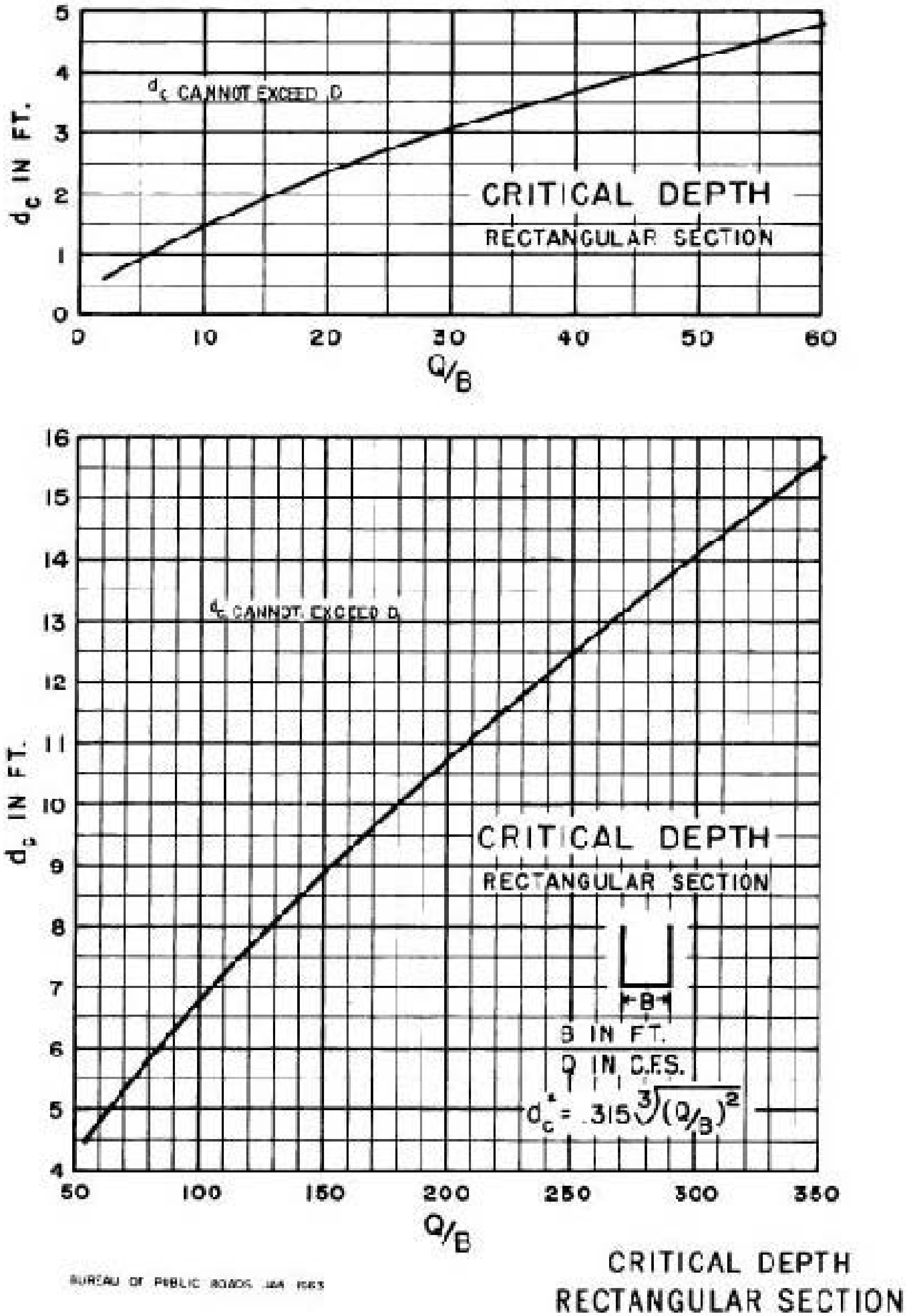
Type of Structure and Entrance Design	ke	Standard Plan
Concrete Pipe		
Projecting from fill, no headwall		
Socket end (groove end)	0.2	
Square cut end	0.5	
Mitered to conform to fill slope (beveled end section)	0.7	B-7a
Mitered to conform to fill slope, with concrete headwall	0.7	B-9
Flared end sections, metal or concrete	0.5 B-7	Design B
Vertical headwall with wingwalls		
Socket end (groove end)	0.2 B	
Square cut end	0.5	
Rounded (radius = 1/12 D)	0.2*	
Metal and Thermoplastic Pipe or Pipe Arch		
Projecting from fill, no headwall	0.9	
Tapered end section	0.9	B-9c, B9-d
Mitered to conform to fill slope (beveled end section)	0.7	B-7a
Mitered to conform to fill slope, with concrete headwall	0.7	B-9
Flared metal or thermoplastic end sections	0.5 B-7	Design A
Vertical headwall with wingwalls	0.5	
Any headwall with beveled inlet edges	0.2*	
Reinforced Concrete Box		
Mitered concrete headwall to conform to fill slope		
Square-edged on 3 edges	0.5	
Rounded or beveled edges on 3 sides	0.2	
Wingwalls at 30 degrees to 75 degrees to barrel		
Square edge at crown	0.4	
Rounded or beveled edge at crown	0.2*	
Wingwalls at 10 degrees to 25 degrees to barrel		
Square edge at crown	0.5	
Wingwalls parallel to barrel		
Square edge at crown	0.7	
Side or slope tapered inlet	0.2*	

*Reference Section 3-4.6 for the design of special improved inlets with very low entrance losses
**Modified for round pipe.

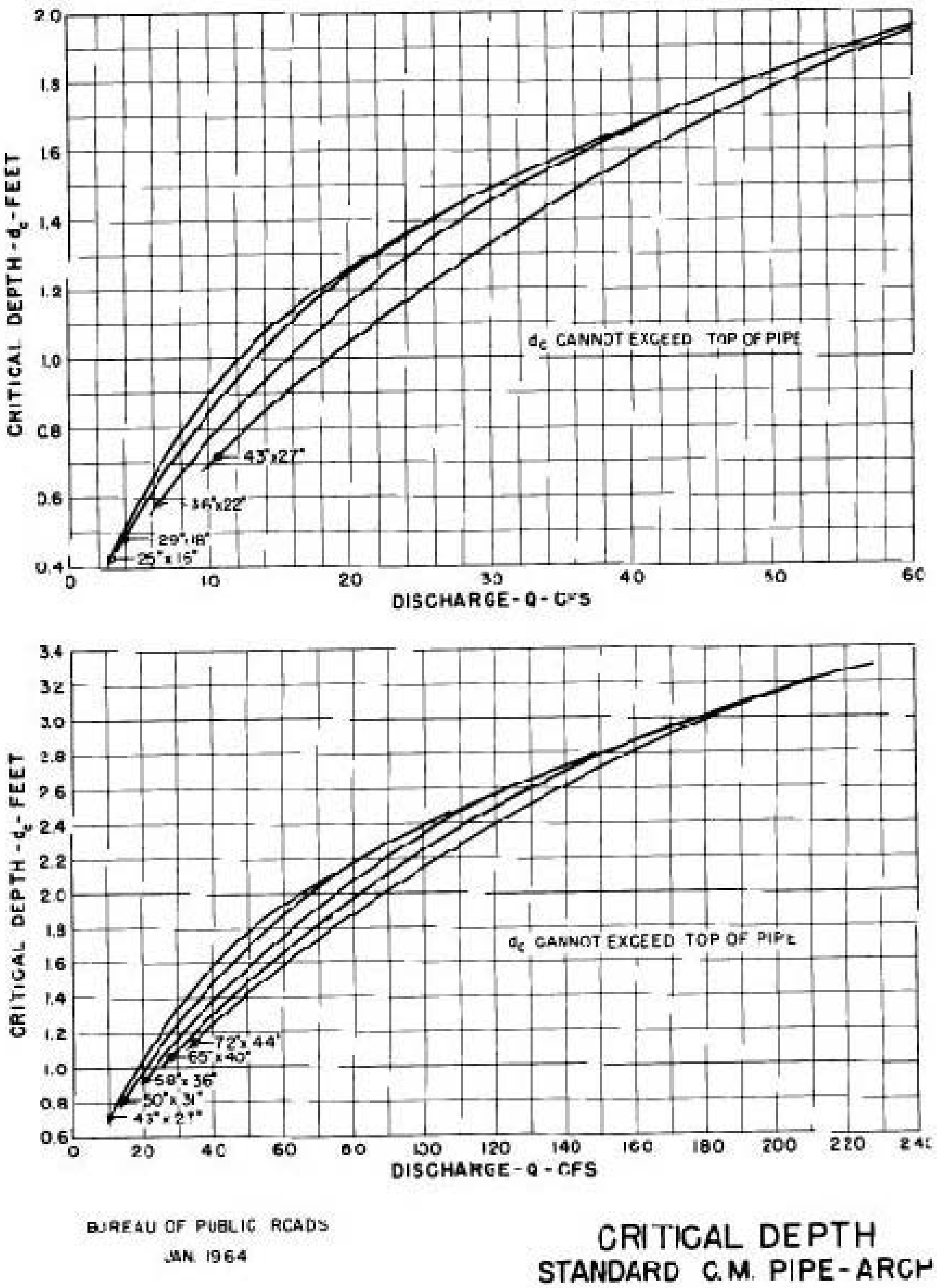
Entrance Loss Coefficient k_e Outlet Control
Figure 3-3.4.5H



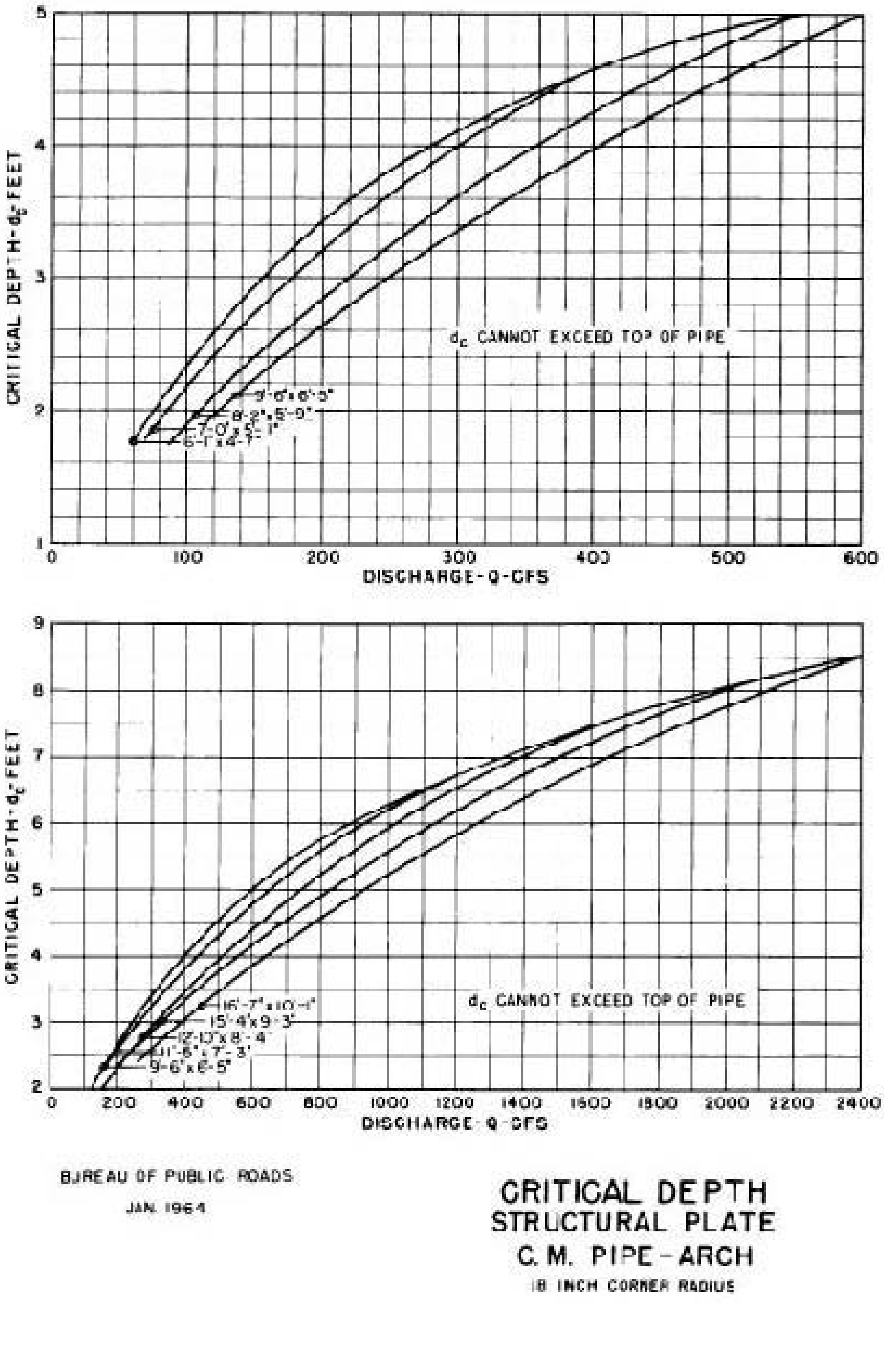
Critical Depth for Circular Pipe
Figure 3-3.45I



Critical Depth for Rectangular Shapes
Figure 3-3.4.5J



Critical Depth for Standard Corrugated Metal Pipe Arch
Figure 3-3.4.5K



Critical Depth for Structural Plate Corrugated Metal Pipe Arch
Figure 3-3.4.5L

3-3.5 Velocities in Culverts — General

A culvert, because of its hydraulic characteristics, generally increases the velocity of flow over that in the natural channel. High velocities are most critical just downstream from the culvert outlet and the erosion potential from the energy in the water must be considered in culvert design.

Culverts that produce velocities in the range of 1 to 3 m/s (3 to 10 ft/s) tend to have fewer operational problems than culverts that produce velocities outside of that range. Varying the grade of the culvert generally has the most significant effect on changing the velocity, but since many culverts are placed at the natural grade of the existing channel, it is often difficult to alter this parameter. Other measures, such as changing the roughness characteristics of the barrel, increasing or decreasing the culvert size, or changing the culvert shape should be investigated when it becomes necessary to modify the outlet velocity.

If velocities are less than about 1 m/s (3 ft/s), siltation in the culvert may become a problem. In those situations, it may be necessary to increase the velocity through the culvert or provide a debris basin upstream of the inlet. A debris basin is an excavated area upstream of the culvert inlet that slows the stream velocity and allows sediments to settle out prior to entering the culvert. See Section 3-8.4 for additional information on debris basins. If the velocity in the culvert cannot be increased and if a debris basin cannot be provided at a site, another alternative is to provide oversized culverts. The oversized culverts will increase siltation in the culvert, but the larger size may prevent complete blocking and will facilitate cleaning. It is recommended that the designer consult with the Region Hydraulics Section/Contact to determine the appropriate culvert size for this application.

If velocities exceed about 3 m/s (10 ft/s), abrasion due to bed load movement through the culvert and erosion downstream of the outlet can increase significantly. Abrasion is discussed in more detail in Section 8-6. Corrugated metal culverts may be designed with extra thickness to account for possible abrasion. Concrete box culverts and concrete arches may be designed with sacrificial steel inverts or extra slab thicknesses to resist abrasion. Adequate outlet channel or embankment protection must be designed to insure that scour holes or culvert undermining will not occur. Energy dissipators can also be used to protect the culvert outlet and downstream property, as discussed in Section 3-4.7. The designer is cautioned that energy dissipators can significantly increase the cost of a culvert and should only be considered when required to prevent a large scour hole or as remedial construction.

3-3.5.1 Calculating Outlet Velocities for Culverts in Inlet Control

When a culvert is flowing in inlet control, the water surface profile can be assumed to converge toward normal depth as flow approaches the outlet. The average outlet velocity for a culvert flowing with inlet control can be approximated by computing the normal depth and then the normal velocity for the culvert cross-section using Manning's equation, as shown below.

The normal depth approximation is conservative for short culverts and close to actual for long culverts. When solving for velocity using computer programs, a different velocity will be obtained. This occurs because the program does not make the normal depth approximation but rather computes a standard step backwater calculation through the pipe to develop the actual depth and velocity. The following equation is for full flow (80% to 100%):

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(English Units)}$$

or

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \text{(Metric Units)}$$

Where: V = Mean full velocity in channel, m/s (ft/s)

n = Mannings roughness coefficient (see Appendix 4-1)

S = Channel slope, m/m (ft/ft)

R = Hydraulic radius, m (ft)

A = Area of the cross section of water, m^2 (ft^2)

P = Wetted perimeter, m (ft)

Manning's equation should be used to solve for the outlet velocity in non-circular culverts. The procedure for determining the velocity is discussed in Chapter 4-3.

For circular culverts, a simplified version of Manning's equation can be used to calculate the velocity in the culvert. The simplified equation for partial flow (10%-80%) is given by:

$$V_n = \frac{0.863 S^{0.366} Q^{0.268}}{D^{0.048} n^{0.732}}$$

Where: S= Pipe slope (ft/ft)

Q = Flow rate (cfs)

D = Pipe diameter (ft)

N = Manning's roughness coefficient

V_n = Normal velocity for partial flow (ft/s)

The above equation was developed from the proportional flow curves shown in Figure 3-3.5.2 and is based on a constant Manning's roughness coefficient. When compared to normal velocities, as calculated by a complete normal depth analysis, the results of this equation are accurate to within ±5 percent.

In some circumstances, a culvert can be flowing in inlet control but the outlet may be submerged. In that situation, the outlet velocity can be found by $V_{out} = Q/A_{total}$, where A_{total} is the full area of the culvert. This condition is rare, and should only be assumed when the outlet is fully submerged and the velocities in the pipe have had a chance to reduce before the outlet.

3-3.5.2 Calculating Outlet Velocities for Culverts in Outlet Control

When a culvert is flowing in outlet control, the average outlet velocity can be found by dividing the discharge by the cross-sectional area of flow at the outlet. There are three general water surface conditions that can exist at the outlet and affect the cross-sectional area of flow. The designer must determine which one of the three conditions exist and calculate the outlet velocity accordingly.

Condition 1: If the tailwater is greater than the diameter of the culvert, the total area of the culvert is used to calculate the outlet velocity.

Condition 2: If the tailwater is greater than critical depth but less than the diameter of the culvert, the tailwater depth is used to calculate the area of flow in the pipe and the corresponding outlet velocity.

In culverts flowing with outlet control, the flow profile tends to converge toward critical depth as flow approaches the outlet. In Condition 2, the flow profile is converging to critical depth near the outlet, but a tailwater depth exists that is greater than the critical depth. Therefore, the tailwater depth will dictate the corresponding area of flow to be used in the velocity calculation.

Condition 3: If the tailwater is equal to or less than critical depth, critical depth is used to calculate the area of flow and corresponding outlet velocity.

Condition 3 represents a situation where a culvert flowing with outlet control is allowed to freely discharge out of the end of the culvert. The tailwater in this case has no effect on the depth of flow at the outlet. Instead, critical depth is used to determine the flow area and corresponding outlet velocity. Critical depth for various shapes can be calculated from the equations shown in Section 4-4 or read from the critical depth charts shown in Figures 3-3.4.5I to L.

Once it has been determined which of the three outlet conditions exist for a given design, the corresponding area of flow for the outlet depth can be determined. The geometrical relationship between the depth of flow and area of flow can range from very simple for structures such as box culverts to very complex for structures such as pipe arches and bottomless culverts. Generally, utilizing a computer program, as discussed in Section 3-3.7, is the most accurate method for completing a culvert design that includes complex shapes.

For circular culverts, the area of flow for a given outlet depth can be determined using the proportional flow curves shown in Figure 3-3.5.2. The curves give the proportional area, discharge, velocity and hydraulic radius of a circular culvert when the culvert is flowing less than full. Once the area has been calculated, the corresponding outlet velocity can be determined. The following example illustrates how to use the chart:

Assume that a design was completed on a 1800 mm (6 ft) diameter pipe with a flow of 4.3 cms (150 cfs). The pipe was found to be in outlet control and a tailwater of 1.5 m (5 ft) was present. Determine the flow condition that exists and calculate the outlet velocity.

Solution:

Step 1 From Figure 3-3.4.5I, critical depth dc was found to be 1.1 m (3.6 ft).

Step 2 Determine the flow condition.

D = 1.8 m (6 ft)

TW = 1.5 m (5 ft)

d_c = 1.1 m (3.6 ft)

Since d_c < TW<D, Condition 2 exists. Therefore, the area of flow caused by the tailwater depth will be used.

Step 3 Find the ratio of the depth of flow (d) to the diameter of the pipe (D), or d/D.

d = tailwater depth = 1.5 m (5 ft)

D = pipe diameter = 1.8 m (6 ft)   d/D = 1.5/1.8 = 0.83

Step 4 Go to the proportional flow curves of Figure 3-3-5.2. Locate 0.83 on the vertical axis. Extend a line horizontally across the page and intercept the point on the "Proportional Area" curve.

Step 5 From the point found on the "Proportional Area" curve, extend a line vertically down the page and intercept the horizontal axis. The value read from the horizontal axis is approximately 0.89. This value represents the ratio of the proportional flow area (A_{prop}) to the full flow area (A_{full}), or A_{prop}/A_{full} = 0.89.

Step 6 Find the proportional flow area. The equation A_{prop}/A_{full} = 0.89 can be rearranged to:

$$A_{prop} = 0.89A_{full}$$

$$A_{full} = \frac{\pi D^2}{4} = \frac{\pi(1.8)^2}{4} = 2.54\ m^2\ (27.3\ ft^2)$$

$$A_{prop} = 0.89(2.54) = 2.26\text{m}^2\ (24.3\text{ft}^2)$$

Step 7 A_{prop} is equal to A_{TW}. Use A_{prop} and Q to solve for the outlet velocity.

$$V_{outlet} = \frac{Q}{A_{prop}} = \frac{4.3}{2.26} = 1.9\frac{m}{s}\ (6.2\frac{ft}{s})$$

The previous example was solved by first determining the proportional area from Figure 3-3.5.2. Utilizing the "Proportional Velocity" curve from the same figure could also have solved the example. Picking up on Step 3 from above, the ratio of d/D would remain the same, 0.83.

Step 4 Go to the proportional flow curves of Figure 3-3.5.2. Locate 0.83 on the vertical axis. Extend a line horizontally across the page and intercept the point on the "Proportional Velocity" curve.

Step 5 From the point found on the "Proportional Velocity" curve, extend a line vertically down the page and intercept the horizontal axis. The value read from the horizontal axis is approximately 1.14. This value represents the ratio of the proportional velocity (V_{prop}) to the full flow velocity (V_{full}), or V_{prop}/V_{full} = 1.14.

Step 6 Rearrange $\frac{V_{prop}}{V_{full}} = 1.14$ to

$$V_{prop} = 1.14V_{full}$$

Step 7 Find V_{full} by solving the equation $V_{full} = \frac{Q}{A_{full}}$

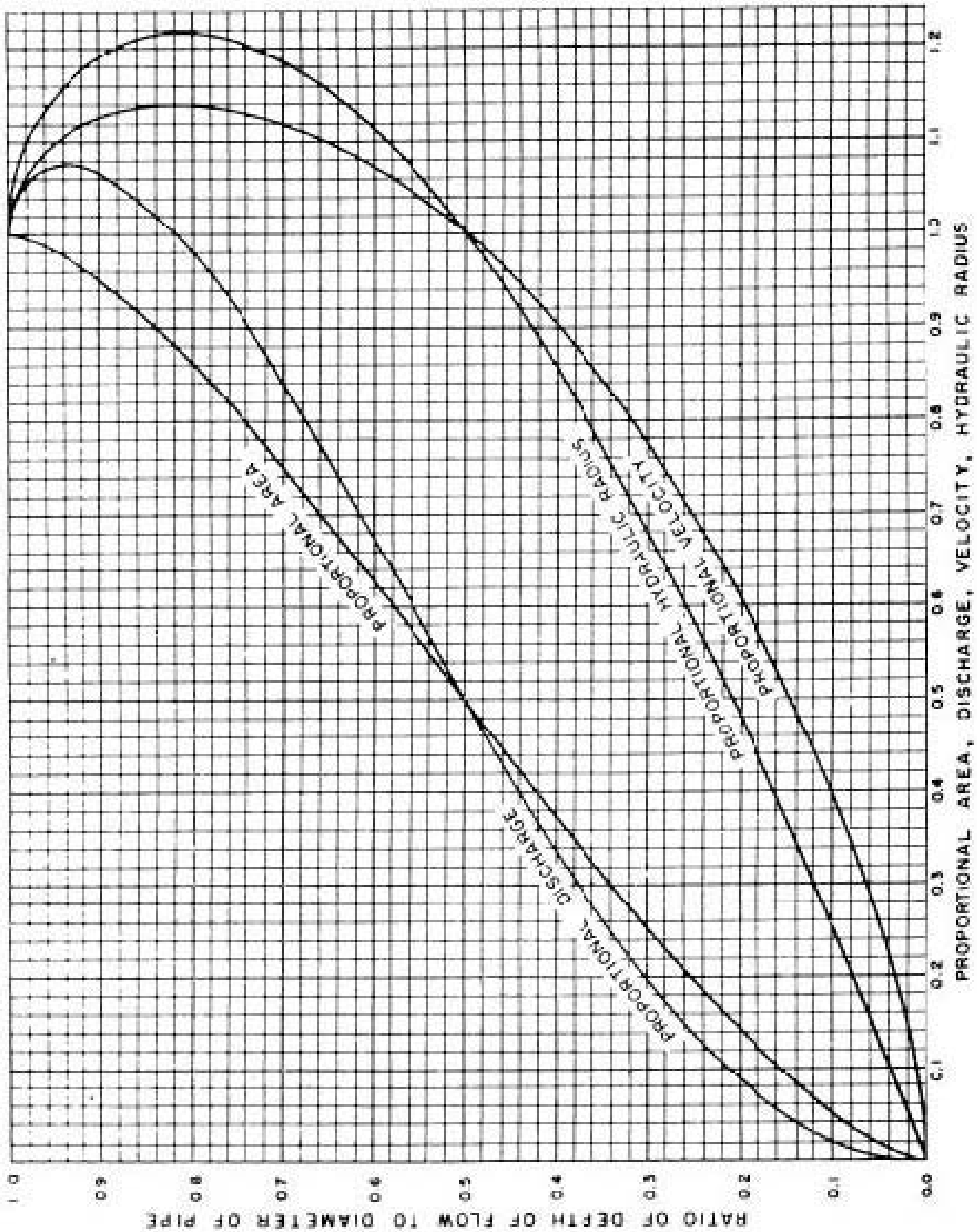
$$Q = 4.3\frac{m^3}{s}$$

$$A_{full} = \frac{\pi D^2}{4} = \frac{\pi(1.8)^2}{4} = 2.54m^2\ (27.3ft^2)$$

$$V_{full} = \frac{4.3}{2.54} = 1.69\frac{m}{s}(5.5\frac{ft}{s})$$

Step 8 Solve for V_{prop}, which is the outlet velocity.

$$V_{prop} = 1.14V_{full} = 1.14(1.69) = 1.9\frac{m}{s}(6.2\frac{ft}{s})$$



Proportional Flow Curve
Figure 3-3-5.2

3-3.6 Culvert Hydraulic Calculations Form

A form "Culvert Hydraulic Calculations" has been developed to help organize culvert hydraulic computations. The form is shown in Figure 3-3.6 and should be used in all Hydraulic Reports that involve culvert designs utilizing hand calculations. If a culvert is designed using a computer program, it is not necessary to include the form in the Hydraulic Report, provided that all design information is included in the input and output files created by the program. Included in this section is an explanation of each of the components of the form and the corresponding chapter section that provides additional information. Figure 3-3.6 has been labeled with either alpha or numeric characters to facilitate discussion for each component of the form. Included after Figure 3-3.6 is a blank copy of the culvert hydraulic calculations form. The blank copy is intended to be used by the designer and included as part of the hydraulic report.

Project: ____________________ Designer: ____________________

SR: ____________________ Date ____________________

Hydrologic and Channel Information

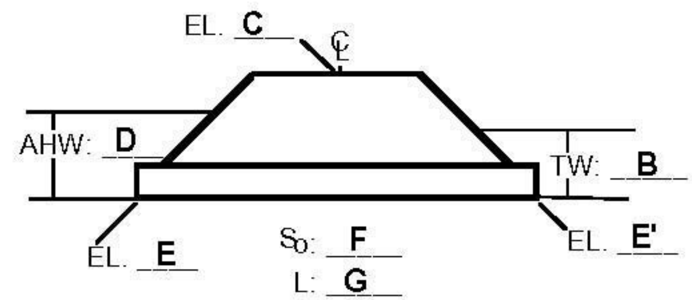
Q_1: A TW$_1$: B

Q_2: A' TW$_2$: B'

Q_3: A" TW$_3$: B"

Sketch

Station: ____________________



			Headwater Computations											
			Inlet Control		Outlet Control									
Culvert Type	Q	Size	$\frac{HW}{D}$	HW	k_e	d_c	$\frac{d_c+D}{2}$	h_0	H	LS_o	HW	Cont. HW	Outlet Vel.	Comments
Column 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15

Summary and Recommendations:

Culvert Hydraulic Calculations Form

Figure 3-3.6

Project: ____________________ Designer: ____________

SR: ____________________ Date ____________

Hydrologic and Channel Information

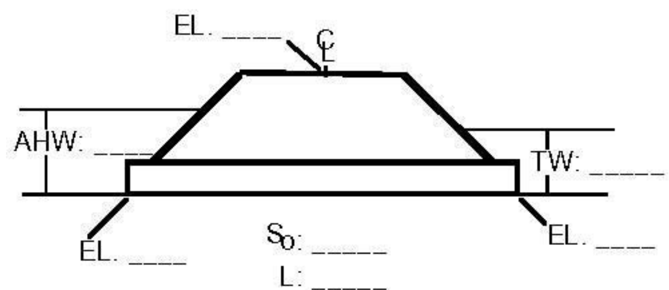
Q_1: ____ TW_1: ____

Q_2: ____ TW_2: ____

Q_3: ____ TW_3: ____

Sketch

Station: ____________



Culvert Type	Q	Size	Headwater Computations									Cont. HW	Outlet Vel.	Comments
			Inlet Control		Outlet Control									
			$\frac{HW}{D}$	HW	k_e	d_c	$\frac{d_c+D}{2}$	h_0	H	LS_o	HW			

Summary and recommendations:

From Figure 3-3.6:

A, A' and A": Design flow(s) Q, in cfs — Section 3-3.1

B, B', and B": Depth of tailwater (TW) in feet, using the corresponding design flow values — Section 3-3.3

C: Elevation of the centerline of the roadway. This is the elevation used to determine roadway overtopping.

D: Allowable headwater depth (AHW), in feet, as discussed in Section 3-3.2 Any significant features upstream that are susceptible to flood damage from headwater should be identified. The elevation at which damage would occur should be identified and incorporated into the design process.

E and E': Inlet and outlet invert elevations, in feet.

F: Slope of culvert (S_o), in feet/feet.

G: Approximate length (L) of culvert, in feet.

Column 1: Culvert Type

Include barrel material, barrel cross-sectional shape, and entrance type.

Column 2: Q — Section 3-3.1

Indicate which design flow from A, A', or A" is being evaluated. Separate calculations must be made for each design flow.

Column 3: Size

Pipe diameter or span and rise, generally indicated in feet.

Column 4: HW_i/D (inlet control)

The headwater to diameter ratio is found from the appropriate nomographs 3-3.4.2A to E.

Column 5: HW (inlet control) — Section 3-3.4.2

This value is found by multiplying Column 3 by Column 4. This is the headwater caused by inlet control. If the inlet control headwater is greater than the allowable headwater as shown in D, the pipe size should be increased. If the headwater is less than allowable, then proceed with the next step. Once the inlet control headwater has been determined, it will be compared with the outlet control headwater in Column 12. The larger of the two values will be the controlling headwater and that value will be entered in Column 13.

Column 6: k_e

This is the entrance loss coefficient for outlet control taken from Figure 3-3.4.5H.

Column 7: Critical Depth

Critical depth can be determined for circular and rectangular shapes by using either the equations shown in Section 4-4 or read from the critical depth charts shown in Figures 3-3.4.5I to L. The critical depth for pipe arches can only be determined by the use of Figures 3-3.4.5K and L.

If critical depth is found to be greater than the pipe diameter or rise, set the critical depth equal to the diameter or rise.

Column 8: $\frac{d_c + D}{2}$ - Figure 3-3.4.4B

The term (d_c + D/2 represents an approximation of the hydraulic grade line at the outlet of the culvert, where d_c is equal to the critical depth at the outlet of the culvert and D is the culvert diameter or rise. It is used to help calculate headwater during outlet control computations. As shown in Figure 3-3.4.4B, (d_c + D)/2 does not represent the actual water surface elevation at the outlet of the culvert and therefore should not be used for determining the corresponding outlet velocity. The method for determining the outlet velocity is discussed in Section 3-3.5.2

Column 9: h_o — Section 3-3.4.4

h_o is equal to either the tailwater or the term (d_c + D)/2, whichever is greater.

Column 10: H — Section 3-3.4.4

H is the total amount of head loss in the barrel of the pipe including the minor losses at the entrance and the exit of the pipe.

The head loss is determined by the equation:

$$H = \left[1 + K_e + \frac{29n^2L}{R^{1.33}}\right]\frac{V^2}{2g}$$

or it may be determined by the outlet control nomographs shown in Figures 3-3.4.5B to G. Both the nomographs and the equation are based on the assumption that the barrel is flowing completely full or nearly full. This is usually the case with most outlet control pipes, but some exceptions do occur. When the barrel is partially full, solving for H using either the nomographs or the equation will tend to overestimate the actual headlosses through the culvert. This will result in a higher, and more conservative, headwater value. A more accurate headwater can be obtained by designing a culvert using a computer program, as described in Section 3-3.7.

Column 11: LS_o

This column is the product of the culvert length (L) multiplied by culvert slope (s_o) or it is equal to the inlet elevation minus the outlet elevation of the culvert.

Column 12: HW— Section 3-3.4.4

This column shows the amount of headwater resulting from outlet control. It is determined by the following equation:

$HW_O = H + h_o - L\ S_o$

Column 13: Controlling HW

This column contains the controlling headwater which is taken from Column 5 or Column 12, whichever is greater. This value is the actual headwater caused by the culvert for the particular flow rate indicated in Column 2.

Column 14: Outlet Velocity

If the culvert was determined to be in inlet control, velocity at the outlet can be determined using the method described in Section 3-3.5.1. If the culvert was determined to be in outlet control, the outlet velocity can be determined using the method described in Section 3-3.5.2.

Column 15: Comments

As appropriate.

Column 16: Summary and Recommendations

As appropriate.

3-3.7 Computer Programs

Once familiar with culvert design theory as presented in this chapter, the designer is encouraged to utilize one of a number one of a number of commercially available culvert design software programs. The Federal Highway Administration has developed a culvert design program called HY-8 that utilizes the same general theory presented in this chapter. HY-8 is menu-driven and easy to use, and the output from the program can be printed out and incorporated directly into the Hydraulic Report. HY-8 is copyright protected but the copyright allows for free distribution of the software. It is available by contacting either the Region Hydraulic Office/Contact or the HQ Hydraulic Branch.

In addition to ease of use, HY-8 is advantageous in that the headwater elevations and outlet velocities calculated by the program tend to be more accurate than the values calculated using the methods presented in this chapter. HY-8 computes an actual water surface profile through a culvert using standard step-backwater calculations. The methods in this chapter approximate this approach but make several assumptions in order to simplify the design. HY-8 also analyzes an entire range of flows input by the user. For example, the program will simultaneously evaluate the headwater created by the $Q_{10\%}$, Q_{25}, and Q_{100} flow events, displaying all of the results on one screen. This results in a significantly simplified design procedure for multiple flow applications.

3-3.8 Example

Note: The example will be presented in English units. The controlling headwater elevations and corresponding outlet velocities will be converted to metric at the conclusion of the design.

A hydrological analysis was completed for a basin above a proposed roadway and culvert crossing. The analysis found that the 25-year flow event was 300 cfs and the 100-year flow event was 390 cfs. In the vicinity of the culvert, the preferable roadway profile would place the centerline at elevation 1,530 feet, about 10 feet higher than the existing channel bottom. The tailwater depth was found to be 5 feet during the 25-year flow event and 5.5 feet during the 100-year flow event. Also, there are no fish passage concerns at this location. Assume that the culvert will be 100 ft long and will match the existing channel slope of 0.005 ft/ft. Then determine the appropriate culvert material and size, and calculate the controlling headwater elevation and corresponding outlet velocity for both the 25- and 100-year events.

Solution

Step 1: The designer must choose an initial type of culvert material to begin the design. Once the culvert is analyzed, the designer may go back and choose a different type of material or pipe configuration to see if the hydraulic performance of the culvert can be improved. In this case, assume that a circular concrete culvert was chosen.

Step 2: Use the hydraulic calculation form shown in Figure 3-3.6 and fill out the known information (see Figure 3-3.8A the complete form for this example). This would include the design flows, tailwater, roadway and culvert elevations, length, slope, and material type. Two design flows were given, one for the 25-year flow event and one for the 100-year flow event. The designer should first analyze the
25-year flow event.

Step 3: The next piece of information needed is the culvert size. In some cases, the culvert diameter is already known and the size can be entered in the appropriate column. In this example, the diameter was not given. In order to determine the appropriate diameter, go to the inlet control nomograph for concrete pipe, Figure 3-3.4.2A.

Step 4: On the nomograph, there are three entrance types available. Assume that in this case, the culvert end will be out of the clear zone and aesthetics are not a concern. Entrance type (3) is an end condition where the pipe is left projecting out of the fill, with the bell or grooved end facing upstream. Choose this entrance type.

Step 5: Because of the relatively low embankment height in this example, it is recommended that the culvert be designed using an HW_i/D ratio during the 25-year event equal to or less than 1.25. On the right hand side of the nomograph, find 1.25 on the vertical HW_i/D scale representing entrance type (3).

Step 6: Using a straightedge, extend that point horizontally to the left and mark the point where it intercepts scale (1). The point marked on scale (1) should be about 1.37.

Step 7: Connect the point just found on scale (1) with 300 cfs on the discharge scale and read the required culvert size on the diameter scale. The value read should be about 75 inches. Since culverts are typically fabricated only in the sizes shown
on the nomograph, choose the next largest diameter available, which in this case is 84 inches (7 feet).

Step 8: The 7-feet diameter culvert is slightly larger than the required size. Therefore, the actual HW_i/D ratio will be less than the 1.25 used to begin the design. To find the new HW_i/D ratio, line up the 84-inch mark on the diameter scale and 300 cfs on the discharge scale, then mark the point where the straightedge intersects scale (1). This value should be about 1.05.

Step 9: Extend that point horizontally to the right to scale (3) and find an HW_i/D ratio of about 0.98. This is the actual HW_i/D ratio for the culvert.

Step 10: Find the inlet control headwater by multiplying the HW_i/D ratio just found by the culvert diameter. $HW = 0.98 \times 7' = 6.86'$. The previous steps found the headwater for inlet control. The next several steps will be used to find the headwater for outlet control.

Step 11: Go to Figure 3-3.4.5H and find the entrance loss coefficient for the culvert. As discussed in Step 4, the grooved end is projecting; therefore, choose an entrance loss coefficient of 0.2.

Step 12: Find the critical depth-using Figure 3-3.4.5I. $d_c = 4.6$ ft

Step 13: Find the value for $(d_c + D)/2$. $(d_c + D)/2 = (4.6 + 7)/2 = 5.8$ ft

Step 14: The value for h_o is equal to the value found for $(d_c + D)/2$ or the tailwater, whichever is greater. In this case, the tailwater was given as 5 ft, therefore, h_o is equal to 5.8 ft.

Step 15: The value for H can be found by using the outlet control nomograph for concrete pipe shown in Figure 3-3.4.5B. With a straightedge, connect the 84-inch point on the diameter scale with the 100-foot length on the 0.2 k_e scale. This will define a point on the turning line. Mark that point.

Step 16: Again with a straightedge, go to the discharge scale and line up 300 cfs with the point just found on the turning line. Extend the line across the page to the head loss scale and find a value of about 1.3 ft.

Step 17: The value for LS_o can be found by multiplying the culvert length times the slope. $LS_o = 100 \times .005 = 0.5$ ft.

Step 18: The outlet control headwater can be found by solving for the equation:

$HW_o = H + h_o - LS_o = 1.3 + 5.8 - 0.5 = 6.6$ ft.

The controlling headwater is the larger value of either the inlet control or the outlet control headwater. In this example, the inlet control headwater was found to be 6.86 feet. This value is greater than the 6.6 ft calculated for the outlet control headwater and therefore will be used as the controlling headwater.

Step 19: Using the equation shown in Section 3-3.5.1, the outlet velocity was found to be 13.2 ft/s. This velocity could cause erosion problems at the outlet, so the designer may want to consider protecting the outlet with riprap, as discussed in Section 3-4.7

The 100-year event must also be checked, using the same procedure. The results of the analysis are summarized below:

HW_i/D:	1.18 ft
HW_i:	8.26 ft
k_e	0.2
d_c	5.1 ft
$(d_c + D)/2$	6.05 ft
h_o	6.05 ft
H	2.2 ft
LS_o	0.5 ft
HW_o	7.75 ft
Cont. HW	8.26 ft
Out. Vel.	14.1 ft/s

Figure 3-3.8A shows a complete culvert hydraulic calculation form for this example. Figure 3-3.8B shows the controlling headwater elevations and outlet velocities for both flow events in English and metric units.

Project:	Designer:
SR:	Date

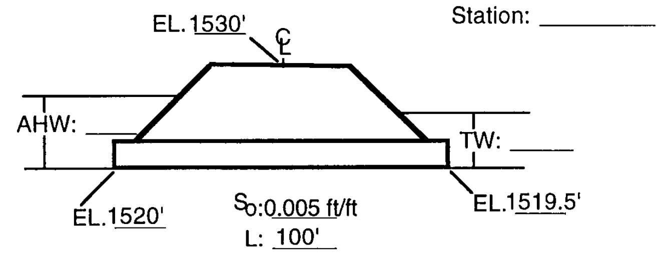
Hydrologic and Channel Information

Q$_1$: 300 cfs   TW$_1$: 5 ft

Q$_2$: 390 cfs   TW$_2$: 5.5 ft

Q$_3$: ____   TW$_3$: ____

Sketch



			Headwater Computations											
			Inlet Control		Outlet Control									
Culvert Type	Q	Size	$\frac{HW_i}{D}$	HW_i	k_e	d_c	$\frac{d_c+D}{2}$	h_0	H	LS_o	HW_o	Cont. HW	Outlet Vel.	Comments
Circ. Concrete	300	7’	0.9	6.86	0.2	4.6	5.80	5.80	1.3	0.5	6.60	6.86	13.2	25-yr, Inlet control
Circ. Concrete	390	7’	1.18	8.26	0.2	5.1	6.05	6.05	2.2	0.5	7.75	8.26	14.1	100-yr, Inlet control

Summary and recommendations: The 100-year headwater is less than 2 feet below the roadway centerline. This may or may not present a problem, depending on the accuracy of the basin flow calculations, the amount of debris in the stream, and the importance of keeping the roadway open during a large event. The designer may want to consider evaluating a different culvert shape, such as a box culvert or low profile arch. These structures tend to provide a larger flow area for a given height, and could potentially pass the design flows without creating as much headwater.

Completed Culvert Hydraulic Calculations Form for Example Problem

Figure 3-3.8A

Flow Event	Controlling Headwater Elevation		Qutlet Velocity	
	m	ft	m/s	ft/s
25-year	465.386	1526.86	4.0	13.2
100-year	465.81	1528.26	4.3	14.1

Example Problem
Figure 3-3.8B

3-4 Culvert End Treatments

The type of end treatment used on a culvert depends on many interrelated and sometimes conflicting considerations. The designer must evaluate safety, aesthetics, debris capacity, hydraulic efficiency, scouring, and economics. Each end condition may serve to meet some of these purposes, but none can satisfy all these concerns. The designer must use good judgment to arrive at a compromise as to which end treatment is most appropriate for a specific site. Treatment for safety is discussed in Section 640.03(4) of the *Design Manual*.

A number of different types of end treatments will be discussed in this section. The type of end treatment chosen for a culvert shall be specified in the contract plans for each installation.

3-4.1 Projecting Ends

A projecting end is a treatment where the culvert is simply allowed to protrude out of the embankment. The primary advantage of this type of end treatment is that it is the simplest and most economical of all treatments. Projecting ends also provide excellent strength characteristics since the pipe consists of a complete ring structure out to the culvert end.

There are several disadvantages to projecting ends. For metal, the thin wall thickness does not provide flow transition into or out of the culvert, significantly increasing head losses (the opposite is true for concrete, the thicker wall provides a more efficient transition). From an aesthetic standpoint, projecting ends may not be desirable in areas exposed to public view. They should only be used when the culvert is located in the bottom of a ravine or in rural areas.

Modern safety considerations require that no projecting ends be allowed in the designated clear zone. See the *Design Manual* (M 22-01) for details on the clear zone and for methods, which allow a projecting end to be used close to the traveled roadway.

Projecting ends are also susceptible to flotation when the inlet is submerged during high flows. Flotation occurs when an air pocket forms near the projecting end, creating a buoyant force that lifts the end of the culvert out of alignment. The air pocket can form when debris plugs the culvert inlet or when significant turbulence occurs at the inlet as flow enters culvert. Flotation tends to become a problem when the diameter exceeds 1800 mm (6 ft.) for metal pipe and 600 mm (2 ft.) for thermoplastic pipe. It is recommended that pipes that exceed those diameters be installed with a beveled end and a concrete headwall or slope collar as described in Sections 3-4.2 and 3-4.4. Concrete pipe will not experience buoyancy problems and can be projected in any diameter. However, because concrete pipe is fabricated in relatively short 2 to 4 meter (6 to 12 ft) sections, the sections are susceptible to erosion and corresponding separation at the joint.

3-4.2 Beveled End Sections

A beveled end treatment consists of cutting the end of the culvert at an angle to match the embankment slope surrounding the culvert. A schematic is shown on Standard Plan B-7a. A beveled end provides a hydraulically more efficient opening than a projecting end, is relatively cost effective, and is generally considered to be aesthetically acceptable. Beveled ends should be considered for culverts about 1800 mm (6 ft.) in diameter and less. If culverts larger than about 1800 mm (6 ft.) in diameter are beveled but not reinforced with a headwall or slope collar, the structural integrity of the culvert can be compromised and failure can occur. The standard beveled end section should not be used on culverts placed on a skew of more than 30 degrees from the perpendicular to the centerline of the highway, however a standard beveled end section can be considered if the culvert is rotated until it is parallel with the highway. Cutting the ends of a corrugated metal culvert structure to an extreme skew or bevel to conform to the embankment slope destroys the ability of the end portion of the structure to act as a ring in compression. Headwalls, riprap slopes, slope paving, or stiffening of the pipe may be required to stabilize these ends. In these cases, special end treatment shall be provided if needed. The Region Hydraulics Section/Contact or the HQ Hydraulics Branch can assist in the design of special end treatments.

3-4.3 Flared End Sections

A metal flared end section is a manufactured culvert end that provides a simple transition from culvert to streambed. Flared end sections allow flow to smoothly constrict into a culvert entrance and then spread out at the culvert exit as flow is discharged into the natural stream or water course. Flared ends are generally considered aesthetically acceptable since they serve to blend the culvert end into the finished embankment slope.

Flared end sections are typically used only on circular pipe or pipe arches. The acceptable size ranges for flared ends, as well as other details, are shown on Standard Plan B-7. Flared ends are generally constructed out of steel and aluminum and should match the existing culvert material if possible. However, either type of end section can be attached to concrete or thermoplastic pipe and the contractor should be given the option of furnishing either steel or aluminum flared end sections for those materials.

A flared end section is usually the most feasible option in smaller pipe sizes and should be considered for use on culverts up to 1800 mm (48 in.) in diameter. For diameters larger than 1800 mm (48 in.), end treatments such as concrete headwalls tend to become more economically viable than the flared end sections.

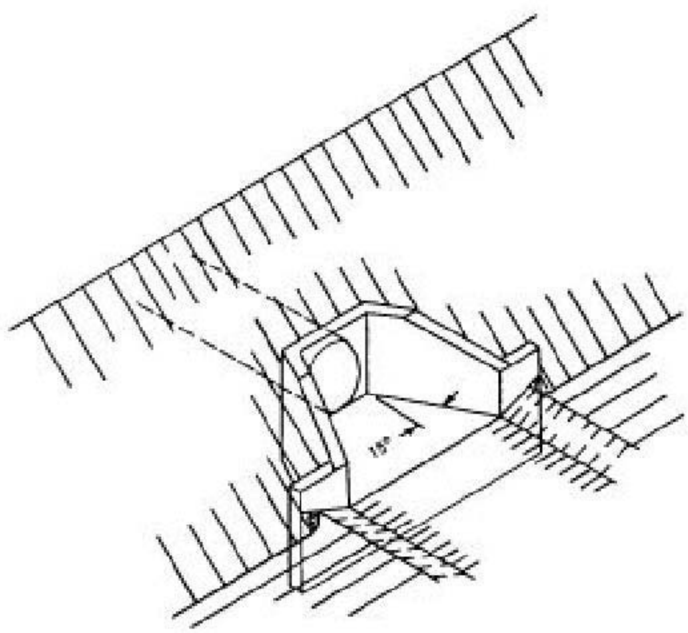
The undesirable safety properties of flared end sections generally prohibit their use in the clear zone for all but the smallest diameters. A flared end section is made of light gage metal and because of the overall width of the structure; it is not possible to modify it with safety bars. When the culvert end is within the clear zone and safety is a consideration, the designer must use a tapered end section with safety bars as shown on Standard Plan B-9c and B-9d. The tapered end section is designed to match the embankment slope and allow an errant vehicle to negotiate the culvert opening in a safe manner.

3-4.4 Headwalls and Slope Collars

A headwall is a concrete frame poured around a beveled culvert end. It provides structural support to the culvert and eliminates the tendency for buoyancy. A headwall is generally considered to be an economically feasible end treatment for metal culverts that range in size from 1800 to 300 mm (6 to 10 ft.). Metal culverts smaller than 1800 mm (6 ft.) generally do not need the structural support provided by a headwall. Headwalls should be used on thermoplastic culverts larger than 600 mm (2 ft.). A typical headwall is shown on Standard Plan B-9. When the culvert is within the clear zone, the headwall design can be modified by adding safety bars. Standard Plan B-9a and B-9b provide the details for attaching safety bars. The designer is cautioned not to use safety bars on a culvert where debris may cause plugging of the culvert entrance even though the safety bars have been designed to be removed for cleaning purposes. When the stream is known to carry debris, the designer should provide an alternate solution to safety bars, such as increasing the culvert size or providing guardrail protection around the culvert end. Headwalls for culverts larger than 3000 mm (10 ft.) tend to lose cost-effectiveness due to the large volume of material and forming cost required for this type of end treatment. Instead, a slope collar is recommended for culverts larger than 3000 mm (10 ft.). A slope collar is a reinforce concrete ring surrounding the exposed culvert end. The HQ Hydraulics Branch generally performs the design of the slope during the structural analysis of the culvert.

3-4.5 Wingwalls and Aprons

Wingwalls and aprons are intended for use on reinforced concrete box culverts. Their purpose is to retain and protect the embankment, and provide a smooth transition between the culvert and the channel. Normally, they will consist of flared vertical wingwalls, a full or partial apron, and bottom and side cutoff walls (to prevent piping and undercutting). Wingwalls may also be modified for use on circular culverts in areas of severe scour problems. The apron will provide a smooth transition for the flow as it spreads to the natural channel. When a modified wingwall is used for circular pipe the designer must address the structural details involved in the joining of the circular pipe to the square portion of the wingwall. The HQ Hydraulics Branch can assist in this design.



Modified Wingwall for Circular Pipe
Figure 3-4.5A

3-4.6 Improved Inlets

When the head losses in a culvert are critical, the designer may consider the use of a hydraulically improved inlet. These inlets provide side transitions as well as top and bottom transitions that have been carefully designed to maximize the culvert capacity with the minimum amount of headwater; however, the design and form construction costs can become quite high for hydraulically improved inlets. For this reason, their use is not encouraged in routine culvert design. It is usually less expensive to simply increase the culvert diameter by one or two sizes to achieve the same or greater benefit.

Certain circumstances may justify the use of an improved inlet. When complete replacement of the culvert is too costly, an existing inlet controlled culvert may have its capacity increased by an improved inlet. Improved inlets may also be justified in new construction when the length of the new culvert is very long (over 500 feet) and the headwater is controlled by inlet conditions. Improved inlets may have some slight advantage for barrel or outlet controlled culverts, but usually not enough to justify the additional construction costs. If the designer believes that a particular site might be suitable for an improved inlet, the HQ Hydraulics Branch should be contacted. Also, HDS 5 contains a significant amount of information related to the design of improved inlets.

3-4.7 Energy Dissipators

When the outlet velocities of a culvert are excessive for the site conditions, the designer may consider the use of an energy dissipator. Energy dissipators can be quite simple or very complex, depending on the site conditions. Debris and maintenance problems should be considered when designing energy dissipators. Tyical energy dissipators include:

1. Riprap Protected Outlets

   Riprap placed at the outlet of a culvert is the simplest method of handling outlet velocities when soils are unstable. Section 4-5.1.1 presents a methodology for sizing riprap based on the shear stress of the flow, and this procedure should also be used for sizing riprap at culvert outlets. The depth of flow used in the calculations should be the depth of flow at the outlet of the culvert.

2. Special Energy Dissipating Structures

   Special structures include impact basins and stilling basins designed according to the FHWA Hydraulic Engineering Circular No. 14, “Hydraulic Design of Energy Dissipators for Culverts and Channels.” These structures may consist of baffles, posts, or other means of creating roughness to dissipate excessive velocity. It is recommended that the HQ Hydraulics Branch be consulted to assist in the design of these type of structures.

3. Gabion Revetment

   Using gabion revetment for outlet protection is not recommended, particularly when the outfall is located near a large river. Past flooding experience has shown the river can easily cut below the level of the revetment, causing the design to fail and requiring costly repairs. If a gabion revetment is considered, it is recommended that riprap protection, as described above, be used instead.

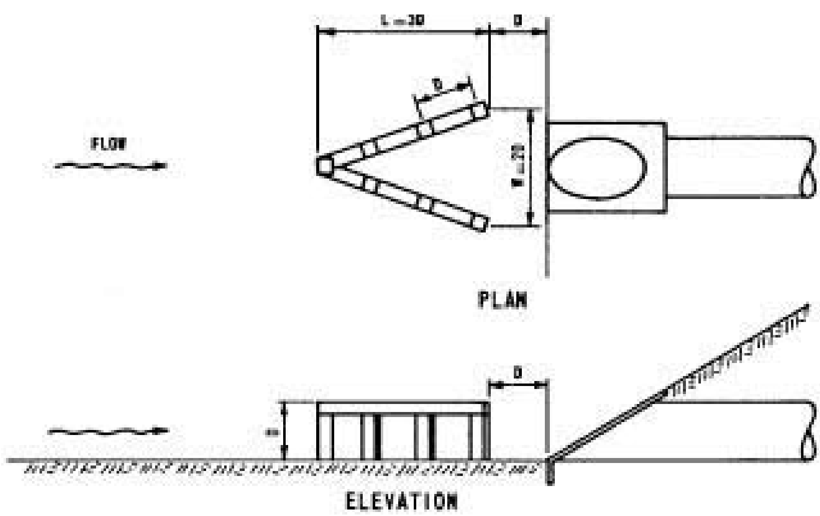
   Energy dissipators have a reputation for collecting debris on the baffles, so the designer should consider this possibility when choosing a dissipator design. In areas of high debris, the dissipator should be kept open and easily accessible to maintenance crews. Provisions should be made to allow water to overtop without causing excessive damage.

3-4.8 Culvert Debris

Debris problems can cause even an adequately designed culvert to experience hydraulic capacity problems. Debris may consist of anything from limbs and sticks or orchard pruning, to logs and trees. Silt, sand, gravel, and boulders can also be classified as debris. The culvert site is a natural place for these materials to settle and accumulate. No method is available for accurately predicting debris problems. Examining the maintenance history of each site is the most reliable way of determining potential problems. Sometimes, upsizing a culvert is necessary to enable it to more effectively pass debris. Upsizing may also allow a culvert to be more easily cleaned. Other methods for protecting culverts from debris problems are discussed below.

1. Debris Deflector (see Figure 3-4.8A)

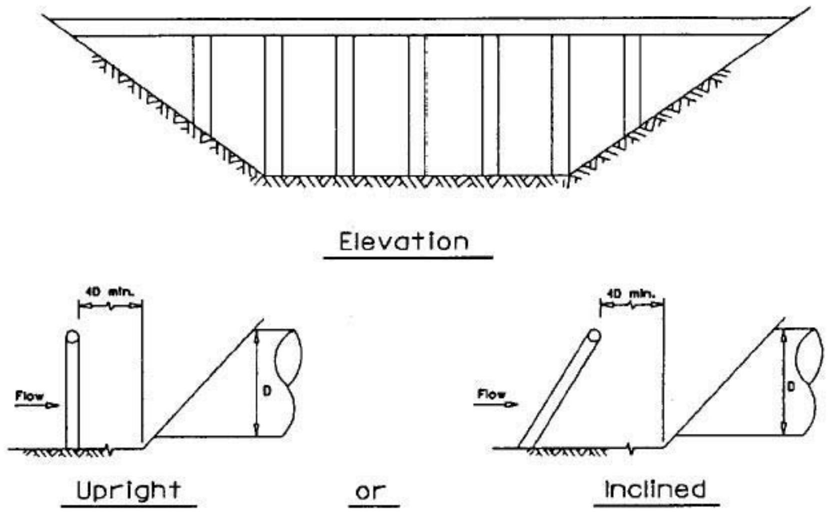
   A debris deflector is V-shaped and designed to deflect heavy floating debris or boulders carried as a bed load in the moderate to high velocity streams usually found in mountains or steep terrain. It is located near the entrance of the culvert with the vortex of the V placed upstream. The horizontal spacing(s) of the vertical members should not exceed “D,” where D is the diameter or the smallest dimension of a non-circular culvert. The length should be 3D, the width 2D, and the height equal to D.



Debris Deflector
Figure 3-4.8A

2. Debris Rack

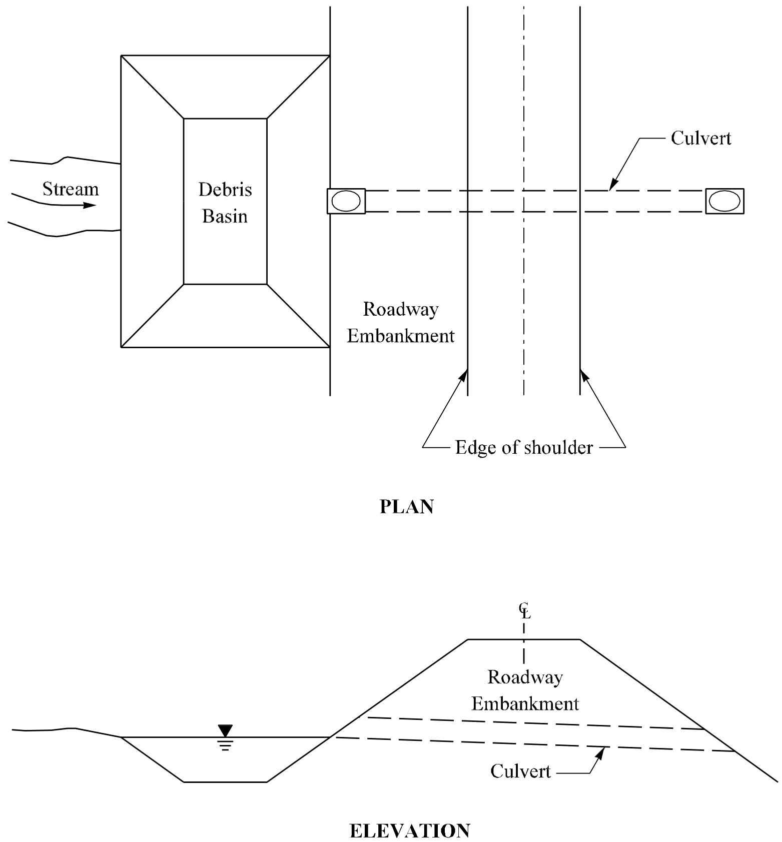
   The debris rack is placed across the channel of the stream. It should be constructed as shown in Figure 3-4.8 B with bars in an upright or inclined position. The bars should be spaced at one-half “D,” where D is the diameter or the smallest dimension of a non-circular culvert. Debris racks should be placed far enough away (approximately 4D) from the culvert entrance so that debris will not block the pipe itself. The debris will frequently become entangled in the rack making removal very difficult, so some thought must be given to placing the rack so it is accessible for necessary maintenance.



Debris Rack
Figure 3-4.8B

3. Debris Basin (see Figure 3-4.8C)

   A debris basin decreases the stream velocity immediately upstream of a culvert inlet, allowing transported sediments to settle out while providing a location for floating debris is collected. A debris basin is generally constructed by excavating a volume of material from below the culvert inlet, as shown in Figure 3-4.8C. The dimensions of a debris basin will vary, depending on the debris history of a site, the potential for future debris, and topographical constraints. It is recommended that the designer consult with the Region Hydraulics Section/Contact to determine the appropriate basin size for a given location. The periodic cleaning of a debris basin is made much easier by providing an access road for maintenance equipment. The cleaning interval needs to be determined from experience depending on the size of the basin provided and the frequency of storms. Debris basins can be quite effective when adequately sized, however, continual maintenance is required regardless of how large they are made.



Debris Basin
Figure 3-4.8C

4. Emergency Bypass Culvert

   In situations where a culvert is placed with a very high fill (over 12 m (40 ft.)) on a stream with significant debris problems, it may be necessary to install an emergency bypass culvert. A plugged culvert in a high embankment can impound a large amount of water. A sudden failure of a high fill is possible, which can result in danger to the downstream property owners and the roadway users. An emergency bypass culvert will limit the level of impounded water to a reasonable amount. The diameter of the bypass culvert should be about 50 percent to 60 percent of the diameter of the main culvert. If possible, the bypass culvert should be placed out of the main flow path so that the risk of it also plugging due to debris is minimized. The invert of the bypass culvert should be placed no more than 5 to 10 feet above the crown of the main culvert, or to the elevation of an acceptable ponding level.

5. Debris Spillway

   Regardless of the efforts made to divert debris from entering a culvert, failures do occur and water could eventually overflow the roadway causing a complete washout of the embankment. The designer should always provide an ample primary culvert system, and in problem areas (e.g., high debris, steep side slopes), some consideration should be given to a secondary or auxiliary drainage facility. This might consist of allowing water to flow over the roadway and spilling over a more stable portion of the embankment without causing complete loss of the embankment.

These spillways should be constructed on, or lined with, material capable of resisting erosion. At some sites the overflow water may have to be directed several hundred feet from its origin in order to find a safe and natural place to spill the water without harm. These secondary drainage paths should always be kept in mind as they can sometimes be utilized at little or no additional cost.

3-5 Miscellaneous Culvert Design Considerations

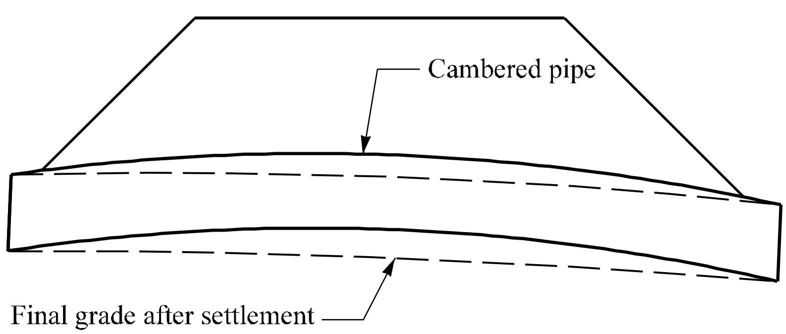
3-5.1 Multiple Culvert Openings

The use of multiple culvert openings is discouraged. It has been observed that this type of system rarely functions as designed because one or more barrels tend to plug with debris. This decreases the effective conveyance capability of the system and can result in failure. Multiple openings have generally been used in situations where very little vertical distance was available from the roadway to the culvert invert. In order to pass the design flow, several identical culverts would have to be placed side by side. New products, such as low profile arches and three-sided box structures, are now available that can provide significant horizontal span lengths while minimizing the necessary vertical rise. The HQ Hydraulics Branch recommends low profile arches or three-sided box structures be considered for use in those type of situations. See Chapter 8 for more information related to arches and three-sided box structures. It is permissible to design a culvert system such that there is a primary conveyance culvert and an emergency bypass culvert placed at a different elevation and to one side of the main channel. This type of design can be effective in situations where significant amounts of woody debris are expected.

3-5.2 Camber

When a culvert is installed under moderate to high fills 10 to 20 m (30 to 60 ft.) or higher, there may be greater settlement of the fill under the center of the roadway than at the sides. This occurs because at the culvert ends there is very little fill while at the centerline of the roadway, the maximum fill occurs. The difference in surcharge pressure at the elevation of the culvert may cause differential settlement of the fill and can create a low point in the culvert profile. In order to correct for the differential settlement, a culvert can be constructed with a slight upward curve in the profile, or camber, as shown in Figure 3-5.2.

The camber is built into the culvert during installation by laying the upstream half of the culvert on a flat grade and the downstream half on a steeper grade in order to obtain the design grade after settlement. The amount of expected camber can be determined by the HQ Materials Lab and must be shown on the appropriate profile sheet in the contract plans.



Camber Under High Fills
Figure 3-5.2

3-5.3 Minimum Culvert Size

The minimum diameter of culvert pipes under a main roadway shall be 18 inches. Culvert pipes from grate inlets or catch basins in the roadway may have a minimum diameter of 12 inches. Culvert pipe under roadway approaches shall have a minimum diameter of 12 inches.

3-5.4 Alignment and Grade

It is recommended that culverts be placed on the same alignment and grade as the natural streambed, especially on year-round streams. This tends to maintain the natural drainage system and minimize downstream impacts.

In many instances, it may not be possible or feasible to match the existing grade and alignment. This is especially true in situations where culverts are conveying only hillside runoff or streams with intermittent flow. If following the natural drainage course results in skewed culverts, culverts with horizontal or vertical bends, or requires excessive and/or solid rock excavation, it may be more feasible to alter the culvert profile or change the channel alignment up or downstream of the culvert. This is best evaluated on a case-by-case basis, with potential environmental and stream stability impacts being balanced with construction and function ability issues.

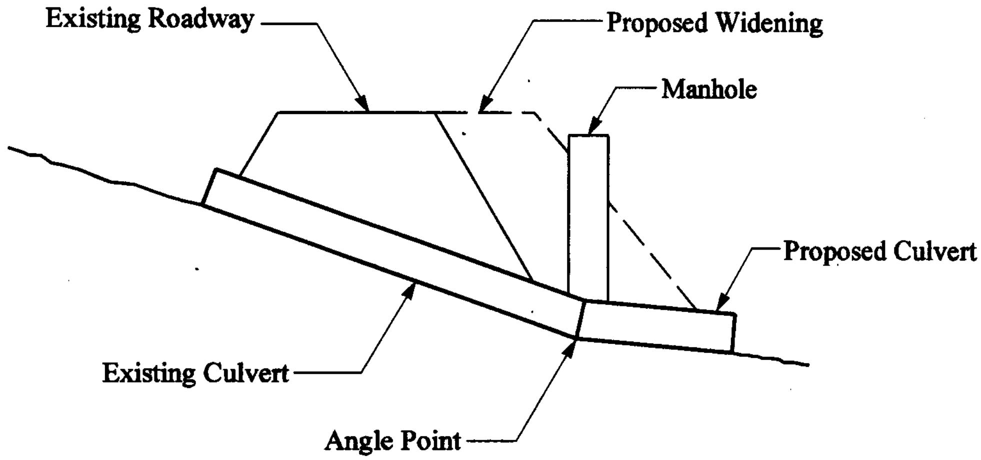
Concrete pipe may be used on any grade up to 10 percent. Corrugated metal pipe and thermoplastic pipe may be used on up to 20 percent grades. For grades over 20 percent, consult with the Region Hydraulics Section/Contact or the HQ Hydraulics Branch for design assistance.

3-5.5 Angle Points

It is recommended that the slope of a culvert remain constant throughout the entire length of the culvert. This is generally easy to accomplish in new embankments. However, in situations where existing roadways are to be widened, it may be necessary to extend an existing culvert at a different slope. The location where the slope changes is referred to as the angle point.

If the new culvert is to be placed at a flatter grade than the existing culvert, it is recommended that a manhole be incorporated into the design at the angle point as shown in Figure 3-5.5A. The change in slope tends to create a location in the culvert that will catch debris and sediment. Providing access with a manhole will facilitate culvert maintenance.

If the new culvert is to be placed at a steeper slope than the existing culvert, the manhole can be eliminated at the angle point if debris and sedimentation have not historically been a concern at the existing culvert.



Culvert Angle Point
Figure 3-5.5

3-5.6 Upstream Ponding

The culvert design methodology presented in Section 3-3 makes the assumption that the headwater required to pass a given flow through a culvert will be allowed to fully develop upstream of the culvert inlet. Any peak flow attenuation provided by ponding upstream of the culvert inlet is ignored. In reality, if a large enough area upstream of the inlet is available for ponding, the design headwater will not occur and the culvert will not pass the full design flow. However, by ignoring any ponding effects, the culvert design is simplified and the final results are conservative. Most culverts should be designed using these assumptions.

If it is determined that the ponding characteristics of the area upstream of the inlet need to be taken into consideration, the calculation of flow becomes a flood routing problem which entails a more detailed study. Essentially, the area upstream of the inlet acts as a detention pond and the culvert acts as an outlet structure. The culvert can be designed utilizing flood routing concepts, but that methodology is beyond the scope of this manual. Since the need for this type of culvert design is rare, the Region Hydraulics Office/Contact or HQ Hydraulics Branch should be contacted for further assistance.

4-1 General

An open channel is a watercourse, which allows part of the flow to be exposed to the atmosphere. This type of channel includes rivers, culverts, stormwater systems that flow by gravity, roadside ditches, and roadway gutters. Open channel flow design criteria are used in several areas of transportation design including:

1. River channel changes.
2. Streambank protection.
3. Partially full-flow culverts.
4. Roadside ditches.
5. Bridge design.
6. Pavement drainage (gutter flow).

Proper design requires that open channels have sufficient hydraulic capacity to convey the flow of the design storm. In the case of earth lined channels or river channels, bank protection is also required if the velocities are high enough to cause erosion or scouring.

The flow capacity of a culvert is often dependent on the channel up and downstream from that culvert. For example, the tailwater level is often controlled by the hydraulic capacity of the channel downstream of the culvert. Knowing the flow capacity of the downstream channel, open channel flow equations can be applied to a typical channel cross section to adequately determine the depth of flow in the downstream channel. This depth can then be used in the analysis of the culvert hydraulic capacity.

Shallow grass lined open channels can contribute to the cleaning of stormwater runoff before it reaches a receiving body. When possible, the designer should route stormwater runoff through open, grass lined ditches, also known as biofiltration swales. When road silts are permitted to settle out, they usually take with them a significant portion of other pollutants. Design criteria for biofiltration swales can be found in Chapter 5 of Washington State Department of Transportation (WSDOT) 2004 *Highway Runoff Manual*.

4-2 Determining Channel Velocities

In open channel flow, the volume of flow and the rate at which it travels are useful in designing the channel. For the purposes of this manual, the determination of the flow rate in the channel, also known as discharge, is based on the continuity of flow equation. This equation states that the discharge (Q) is equivalent to the product of the channel velocity (V) and the area of flow (A).

$$Q = V A$$

Where: Q = discharge, m^3/s (cfs)

V = velocity, m/s (ft/s)

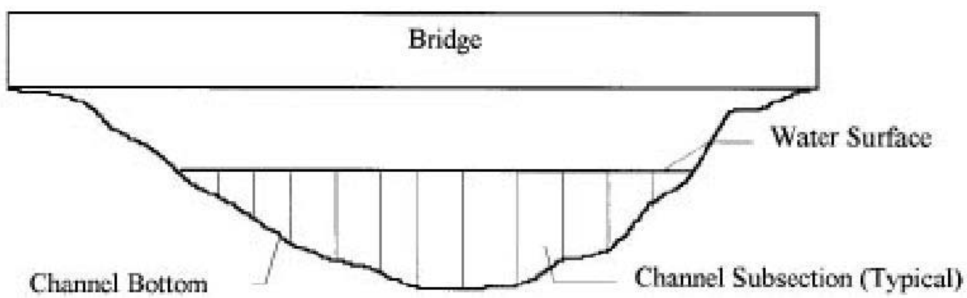
A = flow area, m^2(ft^2).

In some situations, the flow area of a channel is known. If it is not, the flow area must be calculated using an iterative procedure described in Section 4-2.2. Computer programs and charts from FHWA Hydraulic Design Series No. 3 are also available for determining channel geometry or velocities. Channel velocities can either be measured or calculated as described below.

4-2.1 Field Measurement

Because channel velocities are used in determining flow rates, measurements of the channel velocity taken during periods of high flow are of most interest. The designer needs to consider the high flows and ensure that the channel design can provide the required capacity. The velocity can be estimated from field measurements by using one of the following three methods. The first two methods require the use of a current meter. Current meters can measure velocities at any given depth in the channel.

The first method uses surveyed cross sections of the river. At a given cross section, the section is divided into subsections (up to 10 or 20 subsections for best accuracy) as shown in Figure 4-2.1. A change in depth or a change in ground cover is the best place to end a subsection. The current meter is used at each subsection to measure the velocities at 0.2 times the channel depth and at 0.8 times the channel depth. For example, if the channel was only one meter deep in the first subsection, the current meter should be lowered into the water to 0.2 m from the channel bottom and used to read the velocity at this location. The designer would then raise the current meter to 0.8 m from the channel bottom and read the velocity at that location. The velocity of that subsection of the river is the average of these two values. The process is repeated for each of the subsections.



Determining Velocities by Subsections
Figure 4-2.1

Contour maps or surveyed cross sections of the river are also required for the second method. Similar to the first method, the cross section of the river is divided into subsections. However, in the second method, the velocity is only measured at a distance from the channel bottom equivalent to 0.4 times the channel depth. This is considered to be the average velocity for that subsection of the river. A reading is taken at each subsection. This method is slightly less accurate than Method 1 above.

The third method is the least accurate of the three procedures. At the point of interest, the designer should measure the velocity at the surface of the stream. If no current meter is available, throwing a float in the water can do this and observing the time it takes to travel a known distance. The surface velocity is the known distance divided by the time it took to travel that distance. The average velocity is generally taken to be 0.85 times this surface velocity.

Once the velocity of each subsection is measured, the flow rate for each of the subsections is calculated as the product of the area of the subsection and its measured velocity. Summing the flow rates for each subsection will determine the total flow rate, or hydraulic capacity at this cross section of the river.

4-2.2 Manning's Equation

When actual stream velocity measurements are not available, the velocity can be calculated using Manning's Equation. Manning's Equation is an open channel flow equation used to find either the depth of flow or the velocity in the channel where the channel roughness, slope, depth, and shape remain constant (Steady Uniform Flow). The depth of flow using Manning's Equation is referred to as the normal depth and the velocity is referred to as the normal velocity.

The geometry involved in solving Manning's Equation can be complex and consequently, a direct mathematical solution for some channel shapes is not possible. Instead, a trial and error approach may be necessary. Various design tables are available to assist in these solutions as well as several personal computer programs. Information regarding sample programs is available from the Head Quarters (HQ) Hydraulics Branch.

4-2.2.1 Hand Calculations

The solution for flow in an open channel must conform to the following formula:

$$V = \frac{1}{n} R^{2/3} \sqrt{S} \quad \text{(Metric Units)} \qquad V = \frac{1.486}{n} R^{2/3} \sqrt{S} \quad \text{(English Units)}$$

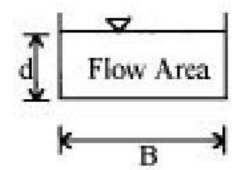
where:

V = Mean velocity in channel, m/s (ft/s)

n = Manning's roughness coefficient (see Appendix 4-1)

S = Channel slope where steady and uniform flow occurs, m/m (ft/ft)

R = Hydraulic radius, m (ft)



R = A/WP

$WP = d + B + d$

A = Area of the cross section of water, m^2 (ft^2)

WP = Wetted perimeter, m (ft)

The hydraulic radius is the ratio of flow area to the wetted perimeter where the wetted perimeter is the length of channel cross section that is in contact with the water. For full flow circular pipes, the hydraulic radius is one-fourth the diameter of the pipe. In relatively flat, shallow channels, where $B > 10d$, the wetted perimeter can be approximated by the width of the channel. As a result, the hydraulic radius can be approximated as the depth of water, $R \simeq d$.

When the depth of flow is known, the mathematical solution is simple. The section properties area (A) and wetted perimeter (WP) can be determined and put into the equation to find velocity (V).

The flow rate, or discharge can then be found by the equation:

$Q = V A$

More frequently, the designer knows the discharge and the depth of flow in the channel must be determined. Since Manning's Equation cannot solve for the depth of a trapezoidal channel directly, a method of successive approximations must be used. The designer must estimate the depth, determine the section properties, and finally solve for the discharge. If the discharge so derived is too high, the designer must then revise the estimated depth downward and recalculate the discharge. This process is repeated until the correct discharge is found within sufficient accuracy (3 to 5 percent). This method can be time consuming. It is recommended that a programmable calculator or computer be used to aid in the computations.

In the following examples, use Figure 4-2.2.1, Geometric Elements of Channel Sections.

Example 1 — A trapezoidal channel with 1V:1.75H side slopes and a 2 m bottom width is flowing 1.2 m deep. The channel has a bottom slope of 0.004 m/m for a distance of several hundred meters. What is the discharge of the riprap lined channel?

Since this is a small channel with riprap, the roughness coefficient of 0.040 is chosen.

$$A = (b + ZD)D = [\,2\text{ m} + 1.75\,(1.2\text{ m})\,]1.2\text{ m} = 4.92\text{ m}^2$$

$$P = b + 2D\ \sqrt{1 + Z^2} = 2\text{ m} + 2(1.2\text{ m})\ \sqrt{1 + 1.75^2} = 6.84\text{ m}$$

$$R = A/P = 4.92\text{ m}^2/6.84\text{ m} = 0.72\text{ m}$$

$$V = \frac{1}{n}\,R^{2/3}\,\sqrt{S} = \frac{1}{0.040}\,(0.72)^{2/3}\,\sqrt{0.004} = 1.27\text{ m/s}$$

$$Q = VA = 1.27\text{ m/s}\,(4.92\text{ m}^2) = 6.25\text{ m}^3/\text{s}\ (220.7\text{ cfs})$$

Example 2 — How deep would the channel described above flow if the discharge is 17 m3/s?

The designer needs to assume various depths and solve for Q as shown above. It may be helpful to draw a graph to aid in choosing the next depth. Once a Q both below and above the discharge, in this case 17 m3/s, is determined the depth can be found using interpolation as shown below.

Assumed D	**Calculated Q**
1.0 m	4.32 m3/s
2.0 m	18.45 m3/s
1.9 m	16.49 m3/s
1.95 m	17.44 m3/s

Interpolate for depth (d) at discharge 17 m3/s:

1. Locate two discharge points, one above and one below 17 m3/s, and note the depth.

   Q = 16.49 m3/s   d = 1.9 m

   Q = 17.44 m3/s   d =1.95 m

2. When interpolation is used, it is assumed that there is a linear relationship between the points. In other words if a straight line was drawn, all 3 points could be located on that line. If there is an unknown coordinate for one of the points, it can be found by finding the slope of the line, as shown below:

$$\frac{(1.95m - 1.90m)}{\left(17.44\frac{m^3}{s} - 16.49\frac{m^3}{s}\right)} = 0.0526\frac{m}{\frac{m^3}{s}}$$

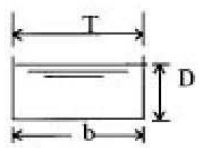
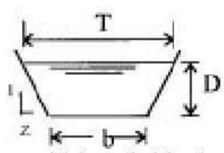
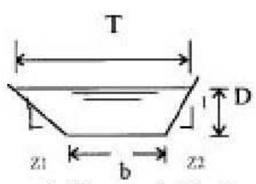
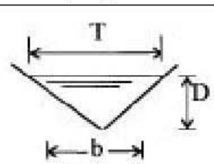
3. Once the slope is known, the depth can be determined at 17 m^3/s:

$$\left(17\frac{m^3}{s} - 16.49\frac{m^3}{s}\right) \times 0.0526\frac{m}{\frac{m^3}{s}} = 0.027m$$

$$d = 1.9m + 0.027m = 1.927m$$

4. Finally, the depth should be verified by rerunning the analysis at Q=17 m^3/s.

Calculations could have been stopped after the third trial, as ± 3 percent is sufficiently accurate for an analysis of this type.

Cross Section	Area, A	Wetted Perimeter, WP
 Rectangle	bD	$b + 2D$
 Trapezoid (equal side slopes)	$(b + ZD)D$	$b + 2D\sqrt{1 + Z^2}$
 Trapezoid (unequal side slopes)	$\frac{D^2}{2}(Z_1 + Z_2) + Db$	$b + D(\sqrt{1 + Z_1^2} + \sqrt{1 + Z_2^2})$
 Triangle	ZD^2	$2D\sqrt{1 + Z^2}$

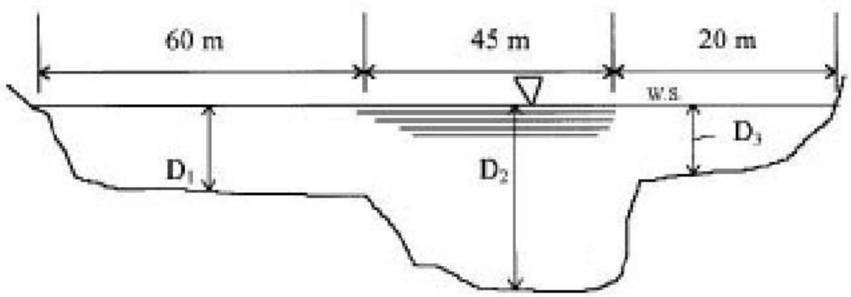
Reference: VT Chow "Open Channel Hydraulics" for a more complete table of geometric elements.

Geometric Elements of Channel Sections

Figure 4-2.2.1

4-2.2.2 Manning's Equation in Sections

Some channel sections are quite variable and may require analysis by subsections. Quite frequently the flow in a rough shallow section may be quite different from the flow in a deep smooth section. Channels and flood plains are a common occurrence of this type. The following example illustrates this situation. Determine the velocity and discharge in each of the three subsections shown below. The river slope is 0.003 m/m.



Subsections Method:	**Section 1**	**Section 2**	**Section 3**
Top Width,	T 60 m	45 m	20 m
Ground Cover	trees	channel	Rock
Manning's Roughness	0.090	0.035	0.060
Flow Depth,	D 1.5 m	4.5 m	1 m
Area, A	90.00 m2	202.50 m2	20.00 m2
Hydraulic Radius,	R 1.50 m	3.93 m	1.00 m
Velocity, V	0.80 m/s	3.90 m/s	0.91 m/s
Discharge, Q	72 m3/s	789.8 m3/s	18.2 m3/s

The total flow rate is equal to the sum of the discharges from each subsection, 880 m^3/s ($3.1x10^4$ cfs) which would be the correct value for the given information. To attempt this same calculation using a constant roughness coefficient, the designer would have to choose between several methods, which take a weighted average of the n-values. Taking a weighted average with respect to the subsection widths or subsection area may appear to be reasonable, but it will not yield a correct answer. The subsection method shown above is the only technically correct way to analyze this type of channel flow. However, this application of Manning's Equation will not yield the most accurate answer. In this situation, a backwater analysis, described in Section 4-4, should be performed. Notice that the weighted average n-value is difficult to choose and that the average velocity does not give an accurate picture as the first method described in Section 4-2.1 Field Measurement.

4-3 Critical Depth

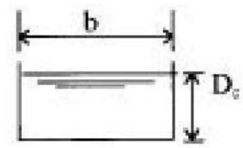
Critical depth is the depth of water at critical flow, which occurs when the specific energy for a given discharge is at a minimum. Critical flow is the dividing point between the subcritical flow regime, where normal depth is greater than critical depth, and the supercritical flow regime, where normal depth is less than critical depth. It is necessary to determine critical depth in the design of open channels because the occurrence of critical depth presents a very unstable condition. A slight change in specific energy, which is the sum of the flow depth and velocity head, could cause a significant rise or fall in the depth of flow. Because of its unstable nature, the facility should be designed to a depth well above or below the critical depth.

Critical flow tends to occur when passing through an excessive contraction, either vertical or horizontal, before the water is discharged into an area where the flow is not restricted. The designer should be aware of the following areas where critical flow could occur: culverts, bridges, and near the brink of an overfall.

A discussion of specific energy is beyond the scope of this manual. The designer should refer to any open channel reference text for further information. Critical depth can be found by the following formulas:

1. Rectangular Channel

$$D_c = \left[\frac{C_1 Q}{b}\right]^{2/3}$$



Where C_1 is 0.319 (metric units) or 0.176 (English units).

Example 3: Find the critical depth in a rectangular channel 4.6 m bottom width and vertical sidewalls. The discharge is 17 m^3/s.

$$D_c = \left[\frac{C_1 Q}{b}\right]^{2/3} = \left[\frac{0.319(17\ m^3/s)}{(4.6\ m)}\right]^{2/3} = 1.12\ m\ \ (3.67\ ft)$$

2. Triangular Channel

$$D_c = C_2\left[\frac{Q}{Z_1 + Z_2}\right]^{2/5}$$

Where C_2 is 0.96 (metric units) or 0.757 (English units).

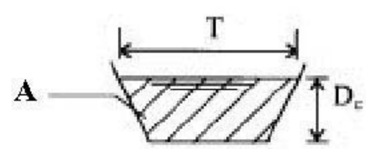
Example 4: Find the critical depth in a triangular shaped channel with 1V:1.75H side slopes for 25 m^3/s.

$$D_c = C_2\left[\frac{Q}{Z_1 + Z_2}\right]^{2/5} = 0.96\left[\frac{25\ m^3/s}{1.75 + 1.75}\right]^{2/5} = 2.11\ m\ \ (6.92\ ft)$$

3. Trapezoidal Channel

depth

$$Q = \left[\frac{gA^3}{T}\right]^{1/2}$$



$$Q = \left[\frac{gA^3}{T}\right]^{1/2}$$

Where "g" is the gravitational constant, 9.81 m/s^2 (metric units) or 32.2 ft/s^2 (English units).

Example 5: Find the critical depth in a trapezoidal channel that has a 3 m bottom width and IV:2H side slopes with 34 m^3/s.

Assume a depth and solve for Q. Repeat process until Q is close to 34 m^3/s.

A programmable calculator is strongly recommended.

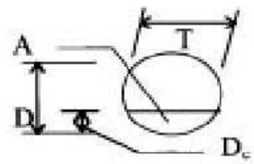
Assumed D (m)	A (m^2)	T (m)	$\left[\frac{gA^3}{T}\right]^{1/2}$
1	5	7	13.24
2	14	11	49.5
1.6	9.29	9.4	31.9
1.65	10.4	9.6	33.9

The critical depth for the given channel and discharge is approximately 1.65 m (5.41 ft).

4. Circular Shaped Channel

Circular channels also require successive approximations for an exact solution.

$$Q = \left[\frac{gA^3}{T}\right]^{1/2}$$



Where "g" is the gravitational constant, 9.81 m/s^2 (metric units) or 32.2 ft/s^2 (English units).

An approximate solution can be found for circular shapes by:

$$D_c = C_3 \frac{Q^{0.5}}{D^{0.25}}$$ where C_3 is 0.562 (metric units) or 0.420 (English units).

Example 6: Find the critical depth for a 1 m diameter pipe flowing with 0.5 m^3/s and for 5 m^3/s.

$$D_c = C_3 \frac{Q^{0.5}}{D^{0.25}} = 0.562 \frac{(0.5 \text{ m}^3/\text{s})^{0.5}}{(1 \text{ m})^{0.25}} = 0.40 \text{ m} \ \ (1.31 \text{ ft})$$

For 5 m^3/s:

$$D_c = C_3 \frac{Q^{0.5}}{D^{0.25}} = 0.562 \frac{(5 \text{ m}^3/\text{s})^{0.5}}{(1 \text{ m})^{0.25}} = 1.26 \text{ m} \ \ (4.13 \text{ ft})$$

Note that 1.26 m is greater than the diameter and therefore has no significance for open channel. The pipe would be submerged and would act as an orifice instead of an open channel.

4-4 River Backwater Analysis

Natural river channels tend to be highly irregular in shape so a simple analysis using Manning's Equation, while helpful for making an approximation, is not sufficiently accurate to determine a river water surface profile. HQ Hydraulics Branch is responsible for computing water surface profiles and has several computer programs to calculate the water surface profile of natural river channels. The computation of the water surface profile is called a backwater analysis.

A backwater analysis is performed when designing a bridge that crosses a river designated as a FEMA regulatory floodway. WSDOT is required by federal mandate to design these bridges to accommodate the 100-year storm event. The water surface elevations for the 100-year and 500-year water surface profiles should be shown on the plans.

In most cases, the construction of a bridge constricts the normal flow patterns of the river. This contraction causes the river to slow and back up upstream of the bridge. Upstream properties could experience some flooding if this backwater is very significant. To minimize the flooding to upstream properties, FEMA regulations are in place to minimize the change in the 100-year water surface elevation between predevelopment and postdevelopment. FEMA regulations only allow up to one-foot increase in the change of the 100-year water surface elevation. The water surface profile must be determined for the before and after developed conditions to verify that this one foot requirement is met. Some local agencies such as King County have more stringent requirements making a detailed computation of the water surface profile even more important.

A backwater analysis can also be useful in the design of culverts. Computing the water surface profile can help the designer determine if the culvert is flowing under inlet, or outlet control. For additional information about backwater analyses, see FHWA's Hydraulic Design Series No. 1, Hydraulics of Bridge Waterways. The region must provide the following information to the HQ Hydraulics Branch to complete a river backwater analysis.

1. A contour map of the project site with 0.25 m (1 ft) or 0.50 m (2 ft) intervals is required. The map should extend from at least one bridge length downstream of the bridge to any point of concern upstream with a minimum distance upstream of two bridge lengths and two meander loops. The map should include all of the area within the 100-year flood plain. All bridge and unique attributes of the project area should be identified.
2. The Manning's roughness coefficients must be established for all parts of the river within the project area. HQ Hydraulics Branch will need photographs of the channel bed and stream bank along the reach of interest to determine the appropriate channel roughness. Photos are especially important in areas where ground cover changes.

To prevent subsequent difficulties in the backwater analysis, the HQ Hydraulics Branch should be contacted to determine the necessary parameters.

4-5 River Stabilization

The rivers found in Washington are still very young in a geological sense and will tend to move laterally across the flood plain from time to time until equilibrium is reached. Whenever a river is adjacent to a highway, the designer should consider the possible impacts of the river on the highway or bridge.

In a natural setting, a river is exposed to several channel characteristics, which help to dissipate some of its energy. Such characteristics include channel roughness, meanders, vegetation, obstructions like rocks or fallen trees, drops in the channel bottom, and changes in the channel cross section. The meander provides an additional length of channel, which allows the river to expend more energy for a given drop in elevation. Vegetation increases the roughness of the channel causing the flow to dissipate more of its energy in order to flow through it. The river utilizes both increased channel length from meanders and increased channel roughness from vegetation to dissipate some of its energy during high runoff periods. When a river overtops its banks, it begins to utilize its flood plains. The flow is either stored in the overbank storage provided by the flood plain or returns to the river downstream. Compared to the flow in the river, the flow returning to the river has been slowed significantly due to the increased roughness and travel length.

Inevitably, roadways are found adjacent to rivers because roadway construction costs are minimized when roadways are constructed through level terrain. At times, roadways built in the flood plain confine the river to one side of the roadway, reducing its channel length. At other times, rivers are confined to their channel to minimize flooding of adjacent properties. As a result, rivers are unable to utilize overbank storage areas. These two situations produce rivers that are highly erosive because the river can no longer dissipate the same amount of energy that was dissipated when the river was not confined to a certain area.

These highly erosive rivers have caused significant damages to the state's highways and bridges. Many roadway embankments have been damaged and bridge piers have been undermined, leading to numerous road closures and high replacement costs. Due to the extensive flooding experienced in the 1990s, more attention has been given to stabilizing Washington Rivers and minimizing damages.

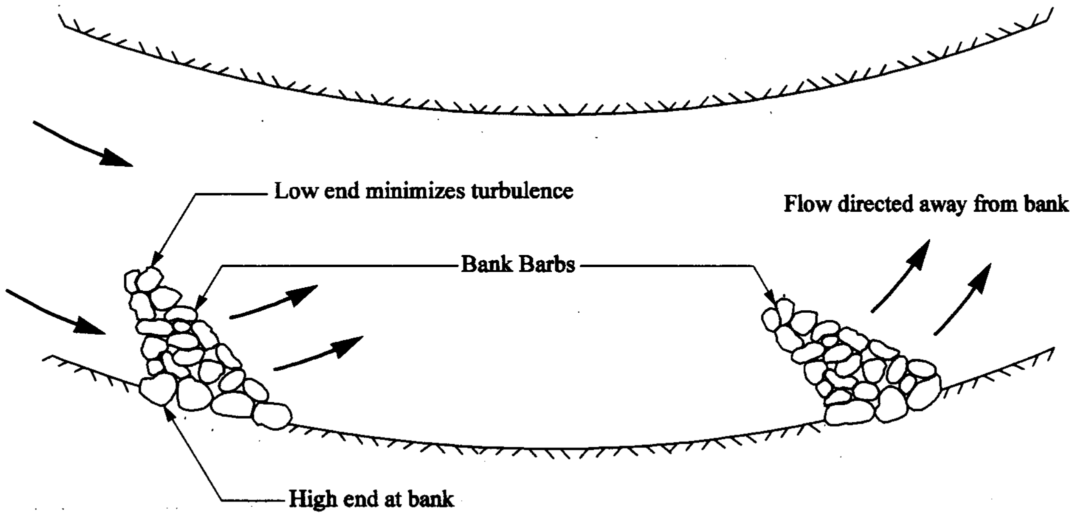
For highly erosive, high-energy rivers, structures constructed in the river's channel are beneficial because they help the river to dissipate some of its energy and stabilize its banks and channel bottom. There are several rock structures that can be used to dissipate energy. Two structures described in the following sections include bank barbs and drop structures. Guide banks and spurs are other examples of in-channel rock structures. Detailed descriptions of guide banks and spurs are provided in the *Hydraulic Engineering Circular No. 20 — Stream Stability at Highway Structures*. When the use of these rock structures is not feasible, riprap bank protection should be used. See Section 4-5.1 and Section 4-5.2 for feasible applications of bank barbs and rock drop structures.

The success of the rock structures or rock bank protection is dependent on the ability of the rock to withstand the forces of the river. As a result, it is of great importance to properly size the rocks used for barbs, drop structures, and bank protection. Although the procedure for sizing the rocks used for barbs and drop structures are similar, riprap sizing for bank protection is not. The methodology for sizing rocks used in each of these structures is described in the individual sections.

For the purposes of this manual, river stabilization techniques include in-channel hydraulic structures only. Bioengineering is the combination of these structures with vegetation, or only densely vegetated streambank projects, which provide erosion control, fish habitat, and other benefits. The designer should consult WSDOT's *Design Manual* Soil Bioengineering Chapter for detailed information about bioengineering.

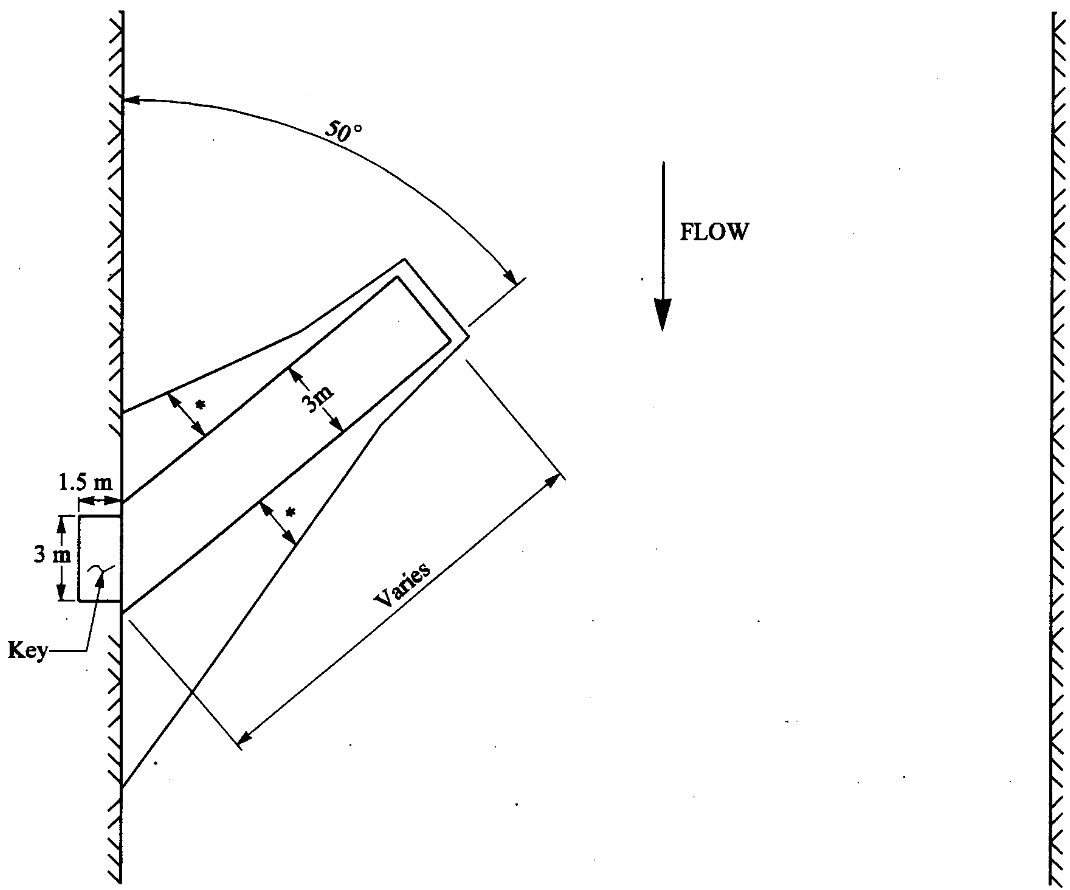
4-5.1 Bank Barbs

Riprap lined channels are very smooth hydraulically. As a result, the river takes the path of least resistance and the deepest part of the channel, or thalweg, is found adjacent to the riprap bank protection. With the thalweg immediately adjacent to the bank protection, scour occurs and the bank protection can be undermined if a toe is not sufficiently keyed into the channel bottom. In this case, it is necessary to shift the thalweg away from the bank and dissipate some of the river's energy to minimize the river's erosive capacity. This can be accomplished by using a bank barb: a trapezoidal shaped rock structure, which extends into the main flow of the river. See Figure 4-5.1.1. Since barbs tend to redirect water to the center of the stream, they encourage deposition between the barbs along the bank.



River Barb Typical Plan View
Figure 4-5.1.1

As shown in Figure 4-5.1.2, the bank barb should extend upstream one-third of the way into the channel at a 50-degree angle. This orientation will capture part of the flow and redirect it perpendicular to the downstream face of the barb. Generally, one barb can protect the length of bank equivalent to about four times the length of the barb perpendicular to the bank. This length of protection is centered about the barb such that two perpendicular barb lengths of bank upstream of the barb and two perpendicular barb lengths of bank downstream of the barb are protected.



River Barb Schematic
Figure 4-5.1.2

The benefits of constructing bank barbs are numerous. The rock structure provides additional roughness to the channel, which slows the flow and helps to decrease its energy. This in turn will reduce the erosive capacity of the river and minimize impacts to roadway embankments and streambanks. They are cost effective since they are less expensive than the alternatives of constructing a wall or placing riprap along a long section of bank. Barbs also provide fish habitat, if habitat features such as logs and root wads are incorporated into the barbs. For more information regarding fish habitat, refer to Chapter 7.

The barbs redirect flow away from the bank minimizing the potential of slope failure. Their ability to redirect the flow can also be useful in training the river to stay within its channel instead of migrating laterally. Minimizing river migration should be considered by the designer when a bridge spans the river. When a bridge is originally constructed, it is designed in such a way that the river flows through the center of the bridge opening. However, after several years, the river will more than likely migrate laterally, possibly endangering bridge piers or abutments because it now flows only along the left side or right side of the opening or it flows at an angle to the bridge. Barbs are an effective tool both training the river to flow through the bridge opening while protecting the bridge abutments.

As effective as barbs are at redirecting flow, there are a few situations where barbs should not be used. For rivers with large bed load (i.e., large quantities of sediments, or large size rocks), barbs may not be as effective at stabilizing the river. Barbs encourage sediments to settle out of the water because they intercept flow and slow it down. If a river has large quantities of sediments, a lot of sediment will tend to settle out upstream and downstream of the barb. The barb will lose its geometric structure and go unnoticed by the river. If the sediments carried downstream by the river are large in size, the barbs could be destroyed from the impact of large rocks or debris.

Barbs may also be ineffective in rivers that flow in a direction other than parallel to the streambank. A barb would not be as effective in this situation because if the flow was at an angle to the streambank, the barb would intercept very little of the flow and thus provide very little redirection.

Three considerations should be taken into account when designing a barb: the size of rock to be used, its placement, and vegetation.

4-5.1.1 Riprap Sizing

The procedure for determining the size of rock needed for a barb can be based on tractive force theory, channel slope, and maximum permissible depth of flow. Tractive force theory is the shear stress exerted by the flow on the channel perimeter, where shear stress is equivalent to the product of channel slope, depth of flow, and the density of water. As any of these factors increase, shear stress increases, and the size of rock necessary to withstand the force of the water will increase. The rock used in the barb must be large enough in both size and weight to resist the force of the water. If the rock is not large enough to withstand the shear stress exerted by the flow, it will be washed downstream.

Assuming that the normal density of water is 9810 N/m3 (62.4 lbf/ft3) and the specific gravity of rock riprap is 2.65, a relation between rock size and shear stress as related to the product of depth times slope is provided below. Once the average channel slope and average depth of flow for the 100-year event is known, the designer can determine the riprap gradation to be used. If the product of slope times flow depth falls between riprap gradations, the larger gradation should be used.

Example 7: Determine the riprap gradation required for a river barb in a reach of river with a channel slope, S = 0.0055 m/m and flow depth, d = 5 m.

S x d = 0.0055 * 5 = 0.0275

From Figure 4-5.1.3, Heavy Loose Riprap should be used. Because the product of slope time's depth, 0.0275, falls between light loose and heavy loose riprap gradations, the larger gradation should be used.

The riprap sizing procedure for bank barbs is not the same procedure used for riprap bank protection. In the case of a barb, the rock is located within the river channel and fully exposed to the flow of the river. The riprap sizing is based on charts relating shear stress to sediment size from *Hydraulic Engineering Circular No. 15 - Design of Roadside Channels with Flexible Linings* and *Hydraulic Engineering Circular No. 11*

- Use of Riprap for Bank Protection. For riprap bank protection, the rock is located along the streambanks with the flow being parallel to the bank. The size of rock required for bank protection will be smaller since its entire surface is not exposed to the flow. Riprap sizing for bank protection is described in Section 4-5.3.1.

Riprap Gradation	D50		Slope Times Flow Depth	
	Metric (m)	English (ft)	Metric (m)	English (ft)
Spalls	0.15	0.5	0.011	0.0361
Light Loose Riprap	0.32	1.1	0.0233	0.0764
Heavy Loose Riprap	0.67	2.2	0.0484	0.1587
1 Meter D50 (Three Man)[1]	1	3.3	0.0721	0.2365
2 Meter D50 (Six Man) [1]	2	6.6	0.1602	0.5256

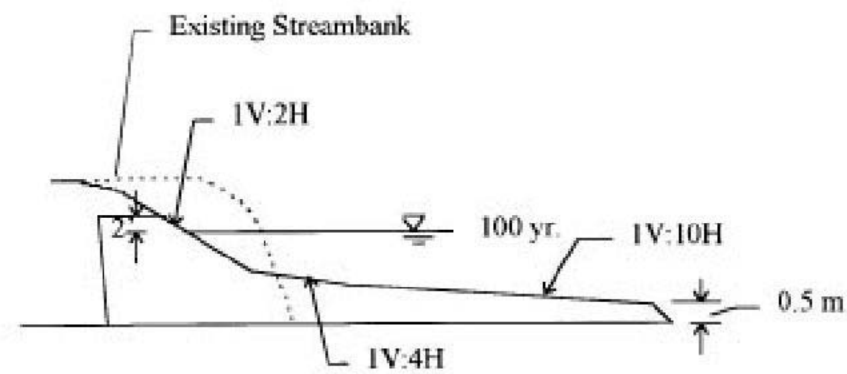
1. See Standard Specification Section 9-13.7(1)

Riprap Sizing for In-channel Structures
Figure 4-5.1.3

4-5.1.2 *Riprap Placement*

When placing the rocks, the larger rocks should be used to construct the base with the rock's longest axis pointed upstream. Smaller rocks can then be used to fill in the voids. The rocks used in the barb must be well graded to ensure interlocking between rocks. The interlocking mechanism is as important as the sizing of the rock. As long as the rocks used in the barb interlock, the barb acts as one entire unit and is better at resisting the shear stress exerted by the flow.

It is essential that the rocks used to form the downstream face are the larger rocks in the riprap gradation and securely set on the channel bottom. The larger rocks along the downstream face provide a base or foundation for the barb as these rocks are subjected to both the forces of the flow and the rocks along the upstream face of the barb. It is also very important to extend a key to the top of the bank or at least two foot above the 100-year flood elevation. See Figure 4-5.1.4. If the flow of water is allowed to get behind the key, the river will take the path of least resistance and the existing streambank that the barb was tied into will erode. The barb will become an ineffective riprap island if not washed downstream.



River Barb Typical Cross Section
Figure 4-5.1.4

4-5.1.3 Vegetation

Vegetation is also a key factor for bank protection. Any land that has been cleared and is adjacent to a river is very susceptible to erosion. Establishing vegetation provides a root system, which can add to the stability of the bank. Plantings also add roughness to the channel slowing the flow. The erosive capacity of the river is reduced for a minimal cost as the energy is dissipated.

The designer should be aware that although vegetation provides some benefits as mentioned above, these benefits are not immediate. There is some risk involved in losing the plantings to a flood before it has time to establish itself and take root. Under favorable conditions, plantings such as willow cuttings and cottonwoods can establish their root systems within a year. Willow cuttings are recommended because of their high survival rate and adaptability to the many conditions specific to typical highway project sites. Cottonwoods are recommended for their extensive root system, which can provide some streambank stability. For detailed information regarding planting type and spacing, the designer should contact the regional landscape architecture office or HQ Roadside and Site Development Services Unit.

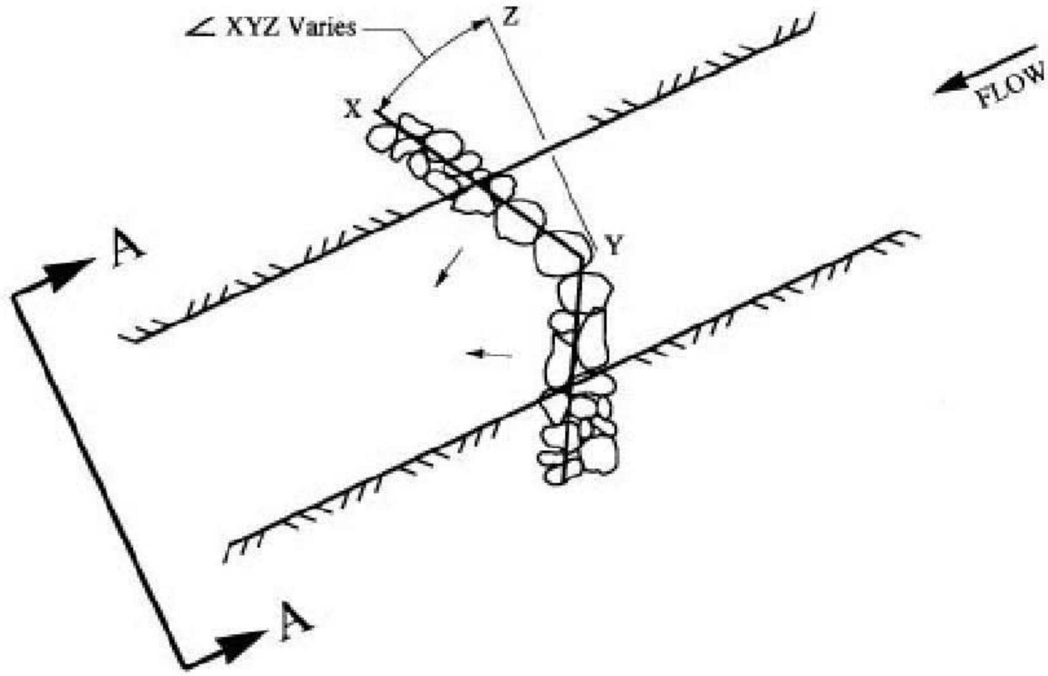
4-5.2 Drop Structures

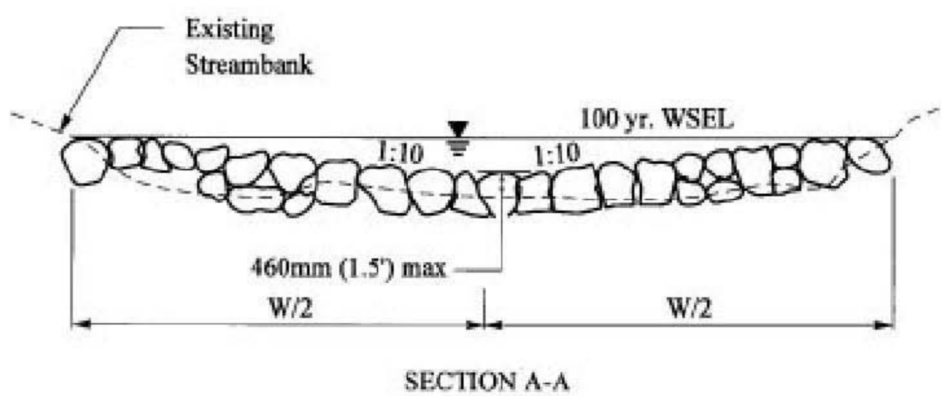
Rock drop structures are very similar to bank barbs in their ability to redirect the flow of the river and decrease its energy. This rock structure redirects the flow towards the center of the channel and is in a V-shape with the V pointing upstream see Figure 4-5.2. As the river flows over the drop structure, the flow is directed perpendicular to the downstream face of the drop structure. However, because of the V-shape of the drop structure, the flow will leave the drop structure in two directions, both aiming towards the middle of the channel. Careful consideration must be given to the angle at which the drop structure is constructed across the river. Substantial scour could be experienced in the middle of the channel if angle XYZ is too large, see Figure 4-5.2.

Two considerations should be taken into account when designing a drop structure: the size of rock and its placement. The procedure for determining the size of rock needed for a drop structure is the same procedure used for river barbs. As a general rule, the size of rock used in the structure should be larger than the size of rocks existing in the bed of the channel. As for the placement of the rock the longest axis of the rock should be pointed upstream. Care should be taken in the height of the drop. The height of the structure should not exceed 1.5 feet (0.5 m) and may be restricted dependent on the species of fish present in the stream. See Chapter 7 or your project biologist for more details. If the drop is too high, a scour hole will form downstream of the base of the structure causing the structure to be undermined and fail.

It is also very important to bury a portion of the drop structure to provide a key into the bank and channel bottom. Similar to barbs, the existing streambank that the drop structure was tied into will erode, if the flow of water is allowed to get behind the key. Specific dimensions of the rock drop structure will be dependent on the river reach of interest. The designer should contact the regional hydraulic staff or HQ Hydraulics Branch for design guidance.

Rock drop structures provide similar benefits as river barbs. In addition to decreasing the energy in the flow and redirecting flow, drop structures like barbs provide some protection for bridge abutments since it is a very effective river training technique.





Drop Structure Plan and Cross Section Views

Figure 4-5.2

Drop structures should be considered when there is a meander propagating toward a bridge. In this case, the river could get behind the bridge abutments and take out the approach fills to the bridge. Unfortunately, meander traits such as location and sinuosity are unpredictable, so unless the bridge spans the entire flood plain, there is no guarantee that the meandering river will not impact the bridge abutment. A drop structure is suitable for this situation because it spans the entire channel and can provide redirection of flow regardless of the direction the intercepted flow is heading.

A barb would not be as effective in this situation because if the flow was at an angle to the streambank, the barb would intercept very little of the flow and thus provide very little redirection. In most cases, the use of drop structures should be limited to smaller, narrow rivers and overflow channels for constructability and permitting reasons. Permitting agencies may not allow construction equipment within the floodway. If the river is too wide, it would be extremely difficult, if not impossible, to set the rocks in the center of the channel with equipment stationed along the bank. The use of drop structures is also discouraged in rivers with large bed load. This structure spans the entire channel and can be damaged when struck by large rocks or woody debris.

4-5.3 Riprap Bank Protection

Similar to barb and drop structures, the tractive force theory can also be applied to bank protection. Tractive force is the shear stress exerted by the flow on the channel perimeter. When the particle size of the native material is too small to resist these tractive forces, erosion will occur and it may be necessary to provide bank protection.

The bank protection provided should be flexible. Riprap bank protection is a good example of a flexible channel lining because the riprap can shift as the bank changes. The rocks are loose and free to move. Rigid channel linings are not recommended for the same reasons that flexible linings are recommended. If rigid linings are undermined, the entire rigid lining as a whole will be displaced increasing the chances of failure and leaving the bank unprotected. Riprap rock encased in grout is an example of a rigid channel lining.

Flexible linings are generally less expensive to install than rigid linings and have self-healing qualities, which reduce maintenance costs. They also permit infiltration and exfiltration and have a natural appearance, especially after vegetation is established.

Riprap bank protection is primarily used on the outside of curved channels or along straight channels when the streambank serves as the roadway embankment. Riprap on the inside of the curve is only recommended when overbank flow reentering the channel may cause scour. On a straight channel, bank protection should begin and end at a stable feature in the bank if possible. Such features might be bedrock outcroppings or erosion resistant materials, trees, vegetation, or other evidence of stability.

4-5.3.1 Riprap Sizing

A design procedure for rock riprap channel linings was developed by the University of Minnesota as a part of a National Cooperative Highway Research Program (NCHRP) study under the sponsorship of the American Association of State Highway and Transportation Officials (AASHTO). The design procedure presented in this section is based on this study and has been modified to incorporate riprap as defined in the WSDOT *Standard Specifications*: Spalls, Light Loose Riprap, and Heavy Loose Riprap.

Manning's Formula or computer programs as previously discussed compute the hydraulic capacity of a riprap-lined channel. The appropriate n-values are shown in Figure 4-5.3.1.

Type of Rock Lining**	N (Small Channels*)	n (Small Channels)	n (Large Channels)
Spalls	D_{50}=0.15m (0.5 ft)	0.035	0.030
Light Loose Riprap	D_{50}=0.32m (1.1 ft)	0.040	0.035
Heavy Loose Riprap	D_{50}=0.67m (2.2 ft)	0.045	0.040

*Small channels can be loosely defined as less than 45 m^3/s (1,500 cfs).

**See the WSDOT Standard Specifications for Road and Bridge Construction Sections 8-15 and 9-13.

Manning's Roughness Coefficients for Riprap (n)
Figure 4-5.3.1

Using Manning's Equation, the designer can determine the slope of the channel, the depth of flow, and the side slopes of the channel required to carry the design flow. The designer, using this information, can then determine the required minimum D_{50} stone size by the following equation:

$$D_{50}=C_R\ d\ S_o$$

Where:

D_{50} = Particle size of gradation, m (ft), of which 50 percent by weight of the mixture is finer
C_R = Riprap coefficient. See Figure 4-5.3.2
d = Depth of flow in channel, m (ft)
S_o = Longitudinal slope of channel, m/m (ft/ft)
B = Bottom width of trapezoidal channel, m (ft). See Figure 4-5.3.2

Channel	**Angular Rock 42° of Repose (0.25' ≤ D_{50} ≤ 3') 0.08m ≤ D_{50} ≤ 0.91m**			**Rounded Rock 38° of Repose (0.25' ≤ D_{50} ≤ 0.75') 0.08m ≤ D_{50} ≤ 0.23m**		
Side Slopes	**B/d=1**	**B/d=2**	**B/d=4**	**B/d=1**	**B/d=2**	**B/d=4**
1:1.5	21	19	18	28	26	24
1:1.75	17	16	15	20	18	17
1:2	16	14	13	17	15	14
1:2.5	15	13	12	15	14	13
1:3	15	13	12	15	13	12
1:4	15	13	12.5	15	13	12.5
Flat Bottom	12.5	12.5	12.5	12.5	12.5	12.5

Note: Angular rock should be used for bank protection because it is better at interlocking and providing a stable slope. Rounded rock coefficients are provided to verify if native material is of sufficient size to resist erosion. Rounded rock usage should be limited to the channel bed region and not used for bank protection purposes because it is not stable. Rounded rock may be used to provide stream bed characteristics in a bottomless arch culvert.

Riprap Coefficients
Figure 4-5.3.2

__Example 8:__ A channel has a trapezoidal shape with side slopes of 1:2 and a bottom width of 3 m. It must carry a Q_{25} = 34 m^3/s and has a longitudinal slope of 0.004 m/m. Determine the normal depth and the type of riprap, if any, that is needed.

n = 0.035

by Manning Formula: d = 2.19 m (7.18 ft)

v = 2.11 m/s (6.93 ft/s)

$$B/d = \frac{3\text{ m}}{2.19\text{ m}} = 1.40 \quad C_R = 15$$

$$D_{50} = C_R\,(d)\,S_o$$

$$D_{50} = 15\,(2.19\text{ m})\,(0.004) = 0.13\text{ m.} \quad (0.43\text{ ft})$$

"Spalls" which has a D_{50} of 0.15 m would give adequate protection to this channel. If the present stream bed has rock which closely matches the calculated D_{50}, then no manmade protection is needed.

Doing the same problem with a 1 percent slope, the designer finds:

n = 0.035
d = 1.76 m (5.77 ft)
V = 2.96 m/s (9.7 ft/s)
B/d = 1.70
C_R = 14
D_{50} = 0.25 m (0.81 ft)

In this case, light loose riprap would be appropriate. The designer may recalculate based on n = 0.040 to get a more exact answer but this would only change the normal depth slightly and would not affect the choice of bank protection. In some cases, on very high velocity rivers or rivers that transport large rocks downstream, even heavy loose riprap may not be adequate to control erosion. Specially sized riprap should be specified in the contract to serve this purpose. HQ Hydraulics Branch and the Materials Lab are available for assistance in writing a complete specification for special riprap.

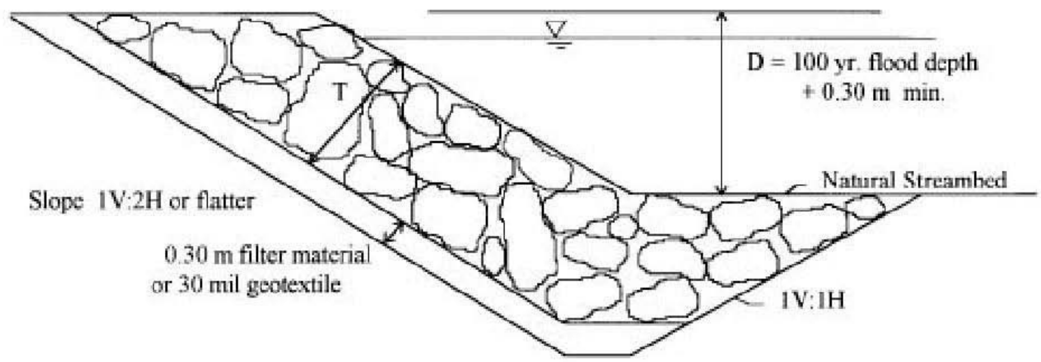
Once the size of riprap is determined, there are several methods in which riprap bank protection can be constructed. Three types of riprap placement including dumped rock riprap, hand-placed riprap and gabions are discussed below.

4-5.3.2 Rock Riprap

Rock riprap is either spalls, light loose, or heavy loose riprap. The riprap should be placed on a filter layer to protect the original bank material from scour or sloughing. The filter layer may consist of on a 1-foot (0.3 m) thick layer of material graded from sand to 6-inch (150-mm) gravel, (placed in layers from fine to coarse out to the riprap). Filter blanket as described in Section 8-15.2 or geotextiles described in Section 9-33 of the *Standard Specifications,* may also be used. In areas of highly erodible soil (fine clay-like soils), HQ Hydraulics should be consulted and an additional layer of sand may be required. No filter layer regardless of type is needed if the existing banks are similar to the filter material of sands and gravels. Riprap thickness is 2 foot (0.6 m) for light loose riprap, 3 feet (0.9 m) for heavy loose riprap, and 1 foot (0.3 m) for quarry spalls. Riprap is usually placed 1 foot (0.30 m) above the 100-year flood depth of the water as shown in Figure 4-5.3.3. This can be made higher if severe wave action is anticipated.

The designer and construction inspectors must recognize the importance of a proper toe or key at the bottom of any riprap bank protection. The toe of the riprap is placed below the channel bed to a depth equaling the toe scour depth. If the estimated scour is minimal, the toe is placed at a depth equivalent to the thickness of the riprap and helps to prevent undermining. Without this key, the riprap has no foundation and the installation is certain to fail. Where a toe trench cannot be dug, the riprap should terminate in a stone toe at the level of the streambed. A stone toe (a ridge of stone) placed along steep, eroding channel banks is one of the most reliable, cost effective bank stabilization structures available. The toe provides material, which will fall into a scour hole and prevent the riprap from being undermined. Added care should be taken on the outside of curves or sharp bends where scour is particularly severe. The toe of the bank protection may need to be placed deeper than in straight reaches.

When a site evaluation or historical evidence indicates that riprap will be needed around a new bridge, the region should indicate this on the Bridge Site Data Sheet (Form 235-001) and refer the riprap design to HQ Hydraulics Branch. See Section 4-5.3.5.



Typical Rock Riprap
Figure 4-5.3.3

4-5.3.3 Hand Placed Riprap

Hand placed riprap is frequently placed around the outlet end of culverts to protect against the erosive action of the water. The size of material at the outlet is dependant on the outlet velocity as noted in Table 4-5.3.3. The limits of this protection would typically cover an area that would normally be vulnerable to scour holes. See Section 3-4.5 for details on wingwalls and aprons.

Outlet Velocity (ft/sec)	Material
6-10	Quarry Spalls
10-15	Light Loose Riprap
>15	Heavy Loose Riprap

Outlet Protection Material Size
Table 4-5.3.3

4-5.3.4 Gabions

Gabions are heavy steel wire mesh containers, rectangular in shape. They vary in size from cubes to mattresses and are filled with clean stone larger than the mesh opening. Gabions can be used in place of other types of riprap due to their flexibility, which allows them to yield to earth movement and still remain efficient and structurally sound. Scour tests have shown that a 0.30 m (1 ft) deep gabion is equal to approximately 0.91 m (3 ft) of riprap.

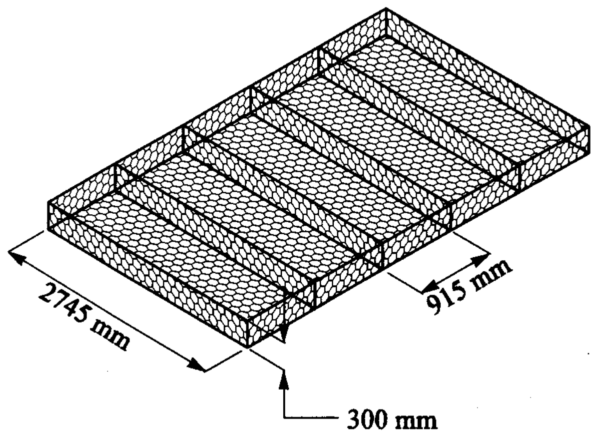
Gabions are generally used when a source of large riprap rock is unavailable, when stream velocities are extremely high, or when space is extremely limited due to steep, narrow streambanks. It is also used in confined spaces such as an existing low bridge, where it is difficult to get the proper size equipment required to place riprap below the bridge. Gabions can be constructed by hand and placed accordingly.

Although gabions are an attractive alternative to using riprap bank protection in the situations mentioned above, the designer should be aware of their drawbacks. Gabions are very labor intensive and require a lot of maintenance if there is a structural breakdown in the steel mesh from the abrasive forces of moving rocks, or from corrosive soils. There may also be some resistance from permitting agencies in the use of gabions because of aesthetic and habitat reasons.

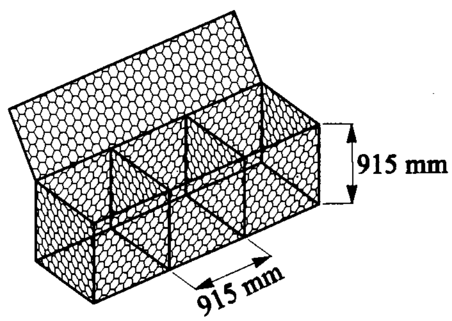
Gabions coated with polyvinyl chloride for the prevention of corrosion are available for salt-water use.

Gabion revetment is not recommended for outlet protection, particularly when the outlet is located near a large river. See section 3-4.7 for more details.

Reference Standard Plan D-6 for further details.



Gabion Mattresses



Gabion Baskets

Gabions
Figure 4-5.3.4

4-5.3.5 Riprap at Bridge Abutments and Piers

Another application of riprap bank protection occurs at bridge abutments and piers. In most cases, bridges over waterways constrict the normal patterns of the river. As a result, bridge abutments and piers are highly susceptible to scour when velocities in the river are high. *FHWA's Hydraulic Circular No. 18, Evaluating Scour at Bridges, Third Edition* is useful when determining anticipated scour depths and adequate protection. If the abutments and piers are to be adequately protected with riprap, the size, gradation, and amount of riprap to be used must be carefully considered. HQ Hydraulics Branch is responsible for calculating the anticipated scour at a bridge and the appropriate riprap protection.

I. Closed Conduits

  A. Concrete pipe 0.010-0.011

  B. Corrugated steel or Aluminum circular pipe or pipe-arch:

    1. 2 2/3 × 1/2 in. Annular Corrugations, treated or untreated 0.022-0.027

    2. 2 2/3 × 1/2 in. Helical Corrugations

      a. Plain or Protective Treatments 1

        (1) 18 inch diameter and below 0.013

        (2) 24 inch diameter 0.015

        (3) 36 inch diameter 0.018

        (4) 48 inch diameter 0.021

        (5) 60 inch diameter 0.022

        (6) 72 inch diameter and above 0.024

      b. Protective Treatments 2 or 4

        (1) 18 inch diameter and below 0.012

        (2) 24 inch diameter 0.014

        (3) 36 inch diameter 0.017

        (4) 48 inch diameter 0.020

        (5) 60 inch diameter 0.021

        (6) 72 inch diameter and above 0.023

      c. Protective Treatments 5 or 6

        (1) All diameters 0.012

    3. 3 × 1 in. Annular Corrugations, treated or untreated 0.027-0.028

    4. 3 × 1 in. Helical Corrugations

      a. Plain or Protective Treatments 1 or 3

        (1) 54 inch diameters and below 0.023

        (2) 60 inch diameter 0.024

        (3) 72 inch diameter 0.026

        (4) 78 inch diameter and above 0.027

      b. Protective Treatments 2 or 4

        (1) 54 inch diameters and below 0.020

        (2) 60 inch diameter 0.021

        (3) 72 inch diameter 0.023

        (4) 78 inch diameter and above 0.024

c. Protective Treatments 5 or 6

(1) All diameters 0.012

5. 5 × 1 in. Annular Corrugations, treated or untreated 0.025-0.026
6. 5 × 1 in. Helical Corrugations

a. Plain or Protective Treatments 1 or 3

(1) 54 inch diameters and below 0.022

(2) 60 inch diameter 0.023

(3) 66 inch diameter 0.024

(4) 72 inch diameter and above 0.025

b. Protective Treatments 2 or 4

(1) 54 inch diameters and below 0.019

(2) 60 inch diameter 0.020

(3) 66 inch diameter 0.021

(4) 72 inch diameter and above 0.022

c. Protective Treatments 5 or 6

(1) All diameters 0.012

C. Steel or Aluminum Spiral Rib Pipe 0.012-0.013

D. Structural Plate Pipe and Plate Pipe Arches 0.033-0.037

E. Thermoplastic Pipe 0.012

1. Corrugated Polyethylene, HDPE 0.018-0.025
2. Profile wall polyvinyl chloride, PVC 0.009-0.011
3. Solid wall polyvinyl chloride, PVC 0.009-0.015

F. Cast-iron pipe, uncoated 0.013

G. Steel pipe 0.009-0.011

H. Vitrified clay pipe 0.012-0.014

I. Brick 0.014-0.017

J. Monolithic concrete:

1. Wood forms, rough 0.015-0.017
2. Wood forms, smooth 0.012-0.014
3. Steel forms 0.012-0.013

K. Cemented rubble masonry walls:

1. Concrete floor and top 0.017-0.022
2. Natural floor 0.019-0.025

L. Laminated treated wood 0.015-0.017

M. Vitrified clay liner plates 0.015

II. Open Channels, Lined (Straight Alignment)

A. Concrete, with surfaces as indicated:

1. Formed, no finish 0.013-0.017
2. Trowel finish 0.012-0.014
3. Float finish 0.013-0.015
4. Float finish, some gravel on bottom 0.015-0.017
5. Gunite, good section 0.016-0.019
6. Gunite, wavy section 0.018-0.022

B. Concrete, bottom float finished, sides as indicated:

1. Dressed stone in mortar 0.015-0.017
2. Random stone in mortar 0.017-0.020
3. Cement rubble masonry 0.020-0.025
4. Cement rubble masonry, plastered 0.016-0.020
5. Dry rubble (riprap) 0.020-0.030

C. Gravel bottom, sides as indicated:

1. Formed concrete 0.017-0.020
2. Random stone in mortar 0.020-0.023
3. Dry rubble (riprap) 0.023-0.033

D. Brick 0.014-0.017

E. Asphalt:

1. Smooth 0.013
2. Rough 0.016

F. Wood, planed, clean 0.011-0.013

G. Concrete-lined excavated rock:

1. Good section 0.017-0.020
2. Irregular section 0.022-0.027

III. Open Channels, Excavated (Straight Alignment, Natural Lining)

A. Earth, uniform section:

1. Clean, recently completed 0.016-0.018
2. Clean, after weathering 0.018-0.020
3. With short grass, few weeds 0.022-0.027
4. In gravelly soil, uniform section, clean 0.022-0.025

B. Earth, fairly uniform section:

1. No vegetation 0.022-0.025
2. Grass, some weeds 0.025-0.030
3. Dense weeds or aquatic plants in deep channels 0.030-0.035

4. Sides clean, gravel bottom 0.025-0.030
5. Sides clean, cobble bottom 0.030-0.040

C. Dragline excavated or dredged:

1. No vegetation 0.028-0.033
2. Light brush on banks 0.035-0.050

D. Rock:

1. Based on design section (riprap) (see 4-6) 0.035
2. Based on actual mean section:
   a. Smooth and uniform 0.035-0.040
   b. Jagged and irregular 0.040-0.045

E. Channels not maintained, weeds and brush uncut:

1. Dense weeds, high as flow depth 0.08-0.12
2. Clean bottom, brush on sides 0.05-0.08
3. Clean bottom, brush on sides, highest stage of flow 0.07-0.11
4. Dense brush, high stage 0.10-0.14

IV. Highway Channels and Swales With Maintained Vegetation *(values shown are for velocities of 2 and 6 fps)*

A. Depth of flow up to 0.7 foot:

1. Bermudagrass, Kentucky bluegrass, buffalograss:
   a. Mowed to 2 inches 0.07-0.045
   b. Length 4 to 6 inches 0.09-0.05
2. Good stand, any grass:
   a. Length about 12 inches 0.18-0.09
   b. Length about 24 inches 0.30-0.15
3. Fair stand, any grass:
   a. Length about 12 inches 0.14-0.08
   b. Length about 24 inches 0.25-0.13

B. Depth of flow 0.7-1.5 feet:

1. Bermudagrass, Kentucky bluegrass, buffalograss:
   a. Mowed to 2 inches 0.05-0.035
   b. Length 4 to 6 inches 0.06-0.04
2. Good stand, any grass:
   a. Length about 12 inches 0.12-0.07
   b. Length about 24 inches 0.20-0.10
3. Fair stand, any grass:
   a. Length about 12 inches 0.10-0.06
   b. Length about 24 inches 0.17-0.09

V. Street and Expressway Gutters

A. Concrete gutter, troweled finish 0.012

B. Asphalt pavement:

1. Smooth texture 0.013

2. Rough texture 0.016

C. Concrete gutter with asphalt pavement:

1. Smooth 0.013

2. Rough 0.015

D. Concrete pavement:

1. Float finish 0.014

2. Broom finish 0.016

3. Street gutters 0.015

E. For gutters with small slope, where sediment may accumulate, increase above values of n by 0.002

VI. Natural Stream Channels

A. Minor streams (surface width at flood stage less than 100 ft):

1. Fairly regular section:

a. Some grass and weeds, little or no brush 0.030-0.035

b. Dense growth of weeds, depth of flow materially greater than weed height 0.035-0.05

c. Some weeds, light brush on banks 0.035-0.05

d. Some weeds, heavy brush on banks 0.05-0.07

e. Some weeds, dense willows on banks 0.06-0.08

f. For trees within channel, with branches submerged at high stage, increase all above values by 0.01-0.02

2. Irregular sections, with pools, slight channel meander; increase values given in 1a-e above 0.01-0.02

3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:

a. Bottom of gravel, cobbles, and few boulders 0.04-0.05

b. Bottom of cobbles, with large boulders 0.05-0.07

B. Flood plains (adjacent to natural streams):

1. Pasture, no brush:

a. Short grass 0.030-0.035

b. High grass 0.035-0.05

2. Cultivated areas:
    a. No crop 0.03-0.04
    b. Mature row crops 0.035-0.045
    c. Mature field crops 0.04-0.05
3. Heavy weeds, scattered brush 0.05-0.07
4. Light brush and trees:
    a. Winter 0.05-0.06
    b. Summer 0.06-0.08
5. Medium to dense brush:
    a. Winter 0.07-0.11
    b. Summer 0.10-0.16
6. Dense willows, summer, not bent over by current 0.15-0.20
7. Cleared land with tree stumps, 100 to 150 per acre:
    a. No sprouts 0.04-0.05
    b. With heavy growth of sprouts 0.06-0.08
8. Heavy stand of timber, a few down trees, little under-growth:
    a. Flood depth below branches 0.10-0.12
    b. Flood depth reaches branches 0.12-0.16

C. Major streams (surface width at flood stage more than 100 ft): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of 0.028-0.033

Reference: *UT Chow "Open Channel Hydraulics" for complete tables and photographs*

5-1 Roadway and Structure Geometrics and Drainage

Roadway and structure pavement drainage should be considered early in the design stage while the roadway geometry is still being developed. The hydraulic capacity of gutters is determined by the longitudinal slope and superelevation of the pavement and minor changes to roadway or structure geometrics are more easily accomplished early in the design phase.

A roadway with a gutter section should normally be placed at a minimum longitudinal slope of 0.3 percent to 0.5 percent to allow for reasonable drainage. The flatter slopes may be used with wider shoulders and the 0.5 percent should be used as a minimum for narrow shoulders. Superelevation and/or widening transitions can create a gutter profile far different from the centerline profile. The designer must carefully examine the geometric profile of the gutter to eliminate the formation of sumps or “ponds” created by these transitions. The areas should be identified and eliminated. This generally requires geometric changes stressing the need for early consideration of drainage.

Improperly placed superelevation transitions can cause serious problems especially on bridges. As discussed in Section 5-4, inlets or other means must pick up gutter flow before the flow crosses over to the other side of the pavement. The collection of crossover flow on bridges is very complex as effective drain inlets are difficult to place within structure reinforcement. Bridges over waterways and wetlands pose water quality issues as well and drop drains may not be allowed. Also, bridge drain downspouts have a history of plugging problems and an objectionable aesthetic impact on the structure.

Eliminating inlets on bridges can usually be accomplished by considering drainage early in the design phase. Superelevation transitions, zero gradients, and sag vertical curves should be avoided on bridges. Modern bridges generally use watertight expansion joints so that all surface water can be run off of the structure and collected in inlets placed at the bridge ends.

Drainage design at bridge ends requires a great deal of coordination between the Region designer, Bridge designer, and the Head Quarters (HQ) Hydraulics Branch. In many areas, the drainage plan may include the bridge. In the past, region responsibility has not included the bridge drainage. The region responsibility ended and began again at the bridge pavement seat. This often results in doubling up of inlets at bridges ends. A more efficient way of handling the drainage is to coordinate efforts. If curb and gutter contain the roadway drainage, and the bridge does not require any drainage, the entire section should be analyzed as a whole.

Multi-lane highways create unique drainage situations. The number of lanes draining in one direction should be considered during the design phase. The Geometric Cross Section chapter in the WSDOT *Design Manual* is a good reference when designing drainage for multi-lane highways. The HQ Hydraulics Branch is also available to provide design guidance. See section 5-6 for discussion on hydroplaning and hydrodynamic drag and how to reduce its potential.

5-2 Computing Runoff for Highway Pavements

The runoff for highway pavements is computed by the rational method (see Chapter 2). This method is easy to use for pavement drainage design because the time of concentration is generally taken as 5 minutes. The recurrence interval for typical pavement drainage design is the 10-year rainfall event. When roadways are placed in a sump, the recurrence interval shall change to the 50-year rainfall event.

5-3 Rural Highway Drainage

When rural highways are built on a fill, roadway drainage is usually allowed to flow, uncollected, to the sides of the roadway and over the side of the fill slope. Usually, this sheet flow of highway drainage does not present any problem to adjacent property owners nor is it a threat to the highway fill. This type of drainage is allowed for fills up to 25 ft (7.5 m). A curb should be used in highly erosive soils when the fill is high enough to justify the use of a guardrail.

Fill heights greater than 25 ft (7.5 m) may present an erosion threat to the embankment especially where the roadway forms a sag vertical curve. This problem is usually present immediately after construction and before vegetation is established. In these situations, it may be prudent to construct a curb and gutter to collect the sheet flow from the pavement and discharge this flow through a water quality or water quantity Best Management Practice (BMP). The treated runoff can then be discharged into an established stream or a low spot in the surrounding terrain. Selection of an appropriate BMP is dependent on the characteristics of the project site. Designers should reference the *Highway Runoff Manual* for selection and design criteria of BMP usage. Some designers prefer to use channels flowing down an embankment to carry away concentrated stormwater. If these channels are lined with rock spalls, they will provide good service for many years. Paved channels, on the other hand, are very vulnerable to damage. The edges of the pavement have been found to break off easily, especially if the capacity of the channel is frequently exceeded or seepage is able to get under the pavement. The HQ Hydraulics Branch does not recommend paved channels unless they have a very short length and have adequate soils supporting the sides of the channel.

An area to be given special attention is at the downstream end of bridges, which generate less flow than necessary to require an inlet and drain pipe. If a storm inlet system is not provided, a channel should be provided at the end of any significant barrier, which collects and concentrates stormwater. Bridges with approach slabs generally have an extruded curb beginning at the bridge end and terminating just past the approach slab. The concentrated flow shall be directed into a rock-lined ditch by creating a small depression and shaping an asphalt chute in the edge of the shoulder apron.

Bridges without approach slabs and curbing pose yet another set of problems. The concentrated flow runs off the bridge slab and flows off the fill slope, or drains behind the wing walls. Care must be taken to assure the flow is directed into the ditch, and not allowed to erode material away from the bridge end.

A ditch running parallel to the roadway generally drains rural highways in a cut section. These ditches are designed and sized in accordance with the criteria shown in Chapters 4 and 7. If the ditch slopes are very steep, they may be fitted with a series of check dams made of rock spalls. Check dams will reduce flow velocities, prevent erosion of the soil, and may help to trap sediment from upstream sources. Check dams as well as other erosion and sediment control BMP's are covered in the *Highway Runoff Manual.*

5-3.1 Slotted Drains and Trench Systems

Historically, slotted drains have been used with varying degrees of success. In fact, the situations, which warrant the use of slotted drain inlets, can actually hinder their performance. Slotted drain inlets are usually placed in areas of minimal horizontal slope and superelevation. Since the invert of the drain is parallel to the pavement, siltation can occur due to low flow velocities.

A number of trench drain systems are available including pre-formed systems, as well as slotted channels that may be attached to metal or polyethylene pipe. The pre-formed systems are constructed of various materials and have a cross section that minimizes siltation. These systems are usually encased in a concrete-backfilled trench that provides the support of the frame. Grates vary depending on application, are produced in a variety of load ratings and may be constructed of ductile iron, stainless or galvanized steel, resin composites or fiberglass.

Other systems consist of slotted channels, usually constructed of metal and may have a minimal slope built in to help minimize the siltation problem. The slotted channel is placed in the pavement, but with the built in slope, the host pipe may be sloped slightly to improve flow. The channels can be attached to metal or polyethylene pipe and come in various widths and lengths. HQ Hydraulics has more information on all these systems and is available to assist in their design.

5-3.2 Drop Inlets

The use of the drop inlet (Standard Plans B-4f thru B-4h) is intended for mountainous areas or portions of highways that have very long continuous grades. They have a high hydraulic capacity. The outlet pipe usually controls the discharge rather than the grate itself. They are also quite effective in passing debris that would normally plug a standard grate. The drop inlet is most often used in medians where safety is a concern. Normal wheel loads can safely pass over the grate and it is not classified as an obstruction.

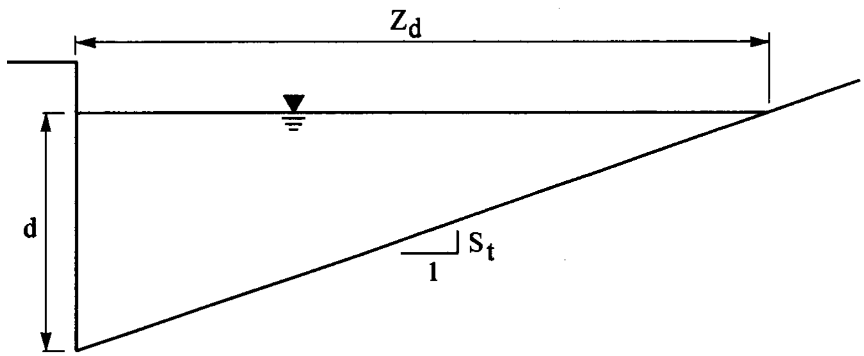
When the inlet is located in the clear zone, the designer should place the inlet as close to parallel to the direction of traffic as possible. Placing the inlet at an angle may cause an errant vehicle to overturn.

5-4 Gutter Flow

On non-interstate roadways where stormwater is collected and carried in a roadside gutter, the top width of the flow prism (Z_d) shall be limited to the shoulder width plus one-half of the traveled lane. On interstate highways with wide shoulders, it is desirable to limit Z_d to the width of the shoulder. In areas where a superelevation transition causes a crossover of gutter flow, the amount of flow calculated at the point of zero superelevation shall be limited to 0.10 cfs (0.003 m^3/s). The designer will find, by the time the roadway approaches the zero point, the Z_d will become very wide. The flow width criteria will be exceeded at the crossover point even when the flow is less than 0.10 cfs (0.003 m^3/s).

The depth of flow should not exceed 0.12 ft (35 mm) at the edge of a traveled lane for non-interstate. In no case should the quantity or velocity of gutter flow be so large as to cause excessive erosion or present a hazard to traffic or pedestrians.

The equation for calculating the gutter flow capacity is a modified version of Manning's equation. It is based on a roughness value of 0.015. Generally, the discharge, longitudinal slope, and superelevation are known and the designer needs to determine the depth of flow and the top width. Using the relationships shown in Figure 5-4, the equations are as follows:



Typical Gutter Section
Figure 5-4

Using metric units,

$$d = \left[\frac{QS_t}{25(S_L)^{0.5}}\right]^{3/8}$$

$$Z_d = \frac{d}{S_t}$$

where:

	d	=	The depth of flow at face of curb (mm)
	Q	=	The gutter discharge (m^3/s)
	S_L	=	The longitudinal slope of the gutter ($^m/_m$)
	S_t	=	The transverse slope or superelevation ($^m/_m$)
	Z_d	=	The top width of the flow prism (m)

Using English units,

$$d = \left[\frac{QS_t}{37(S_L)^{0.5}}\right]^{3/8}$$

$$Z_d = \frac{d}{S_t}$$

where:

	d	=	The depth of flow at face of curb (ft)
	Q	=	The gutter discharge (cfs)

5-5 Catch Basins and Grate Inlets

There are many variables involved in determining the hydraulic capacity of an inlet. The depth of flow next to the curb is the major factor in the interception capacity of an inlet. Slight variations in slope or superelevation can have a large effect on flow patterns. The placement of an inlet can also result in dramatic changes in its hydraulic capacity. Care should be taken to place the inlets next to the face of curb and at the proper elevation relative to the pavement.

Debris floating in the gutter has a tendency to collect at the inlets, plugging part or all of the grate opening. Inlets placed on a continuous grade are calculated using the full width of the grate with no allowance needed for debris. Inlets placed in a sump are calculated using one-half of the open perimeter. Areas with deciduous trees and large pedestrian populations are more prone to debris plugging. Bark from logging operations and agricultural areas are also known to cause debris problems.

To compensate for debris problems, inlets with larger openings may be used such as Grate Inlet Type 1 or 2 (*Standard Plan* B-4b and B-4c), however there are limitations. Neither of these inlets can be installed in heavy traffic areas where wheel loads will pass directly over the grates. The Type 1 inlet is constructed of non-reinforced concrete and cannot support traffic loads and structural failures of both types of grates precludes their use in continuous traffic locations. These inlets may be installed with either a Grate A or B, Grate B has very large openings and is useful in median and ditch locations, areas where there is no pedestrian or bicycle traffic.

Standard Plan B-4d has been tested in H-25 loading and was determined compatible with heavy traffic installations. This frame and double vaned grate should be installed in a Unit H on top of a Grate Inlet Type 2. The frame and grates may be used in either new construction or retrofit situations. . When used in areas of highway speeds, lock down grates should be specified.

The combination inlet (*Standard Plan* B-2e) is a vaned grate on a catch basin with a hooded curb cut area. Its vaned grate is very debris efficient, and if the grate does become clogged, the overflow goes into the hooded opening. These inlets are extremely useful for sag condition installations.

Quarry Spalls shall not be placed around inlets. Quarry spalls have at times been placed around the lids of grate inlets as an erosion control measure. This creates a safety hazard for the maintenance personnel who need good footing to lift the heavy lids. If quarry spalls check dams are desired for erosion control, locate them about 10 feet away from the grate inlet.

		Continuous Grade[1]	Sump Condition [2] Perimeter Flows as Weir	
Standard Plan	**Description**	**Grate Width**	**Width**	**Length**
B-2a[3]	Metal Frame and Grate for Catch Basin and Inlet (Herringbone Pattern)	1.67 ft (0.50 m)	0.69 ft (0.21 m)	0.78 ft (0.24 m)
B-2b	Vaned Grate for Catch Basin and Inlet	1.67 ft (0.50 m)	1.31 ft (0.40 m)	1.25 ft (0.38 m)
B-2e[2]	Combination Inlet	1.67 ft (0.50 m)	1.31 ft (0.40 m)	1.25 ft (0.38 m)
B-4b	Grate Inlet Type 1 (Grate A or B)[4])	2.05 ft (0.62 m)	1.67 ft (0.50 m)	3.52 ft (1.07 m)
B-4c	Grate Inlet Type 2	2.05 ft (0.62 m)	1.67 ft (0.50 m)	3.52 ft (1.07 m)
B-4d	Framed and Vaned Grates for Grate Inlet Type 2	1.71 ft[5] (0.52m) 3.46 ft[6] (1.05 m)	1.29 ft (0.40 m)	1.67 ft (0.50 m)

Properties of Grate Inlets
Figure 5-5

1. Inlet widths on a continuous grade are not reduced for bar area or for debris accumulation.
2. The perimeters and areas in this portion of the table have already been reduced for bar area. These values should be cut in half when used in a sag location as described in Section 5-5.2, except for the Combination Inlet B-2e.
3. Shown for informational purposes only. See Section 5-5.1.
4. Type B grate shall not to be used in areas of pedestrian or vehicular traffic. See section 5-5 for further discussion.
5. Normal Installation, see *Standard Plans*.
6. Rotated Installation see *Standard Plans*.

5-5.1 Capacity of Inlets on a Continuous Grade

The capacity of an inlet on a continuous grade can be found by determining the portion of the gutter discharge directly over the width of the inlet. Research and experience has found that this is a very reasonable estimate of the capacity of the inlet in normal highway applications. This method of calculation is slightly conservative for very flat longitudinal slopes, as side flow interception is ignored, and non-conservative for very steep longitudinal slopes where splash over often occurs. It is most accurate when velocities are in the range of 3 to 5 ft/s (1.0 to 1.5 m/s) at a 2 or 3 percent longitudinal slope. The HQ Hydraulics Branch or FHWA *Hydraulic Engineering Circular No. 22, Section 4-3* is available to assist in the calculation of side flow interception and when velocities exceed 5 ft/s (1.5 m/s).

A Microsoft Excel spreadsheet has been developed that uses the procedure described in this section to calculate roadway runoff and inlet interception for a roadway on a longitudinal slope. The spreadsheets are set up for English or Metric input and output and are located at the web site below. The HQ Hydraulics Office is also in the process of developing two spreadsheets: one that considers side flow interception and one that considers combination inlets (standard plan B-2e). This portion of the manual will be rewritten to include these inlets for the September 2004 revision. If a designer needs the spreadsheets prior to September 2004 they can consult the web site below to see if the spreadsheets are available or contact HQ Hydraulics.

(http://www.wsdot.wa.gov/eesc/design/hydraulics/downloads.htm)

The flow bypassing the first grate inlet shall be calculated and added to the flow intercepted by the second grate located downstream. This carry-over process must continue to the bottom of the grade or the end of the inlet system. The last inlet on a system is permitted to bypass 0.10 cfs (0.003 m^3/s) for the 10 year MRI storm without making any further provisions.

The HQ Hydraulics Branch no longer recommends using herringbone grates. Historically, use of the vaned grate was limited due to cost considerations. The cost difference now is minimal, the vaned grate is bicycle safe, and as described below is hydraulically superior under most conditions. Installation of the vaned grate is critical as the grate is directional. If installed backwards the interception capacity is severely limited. Figure 5-5 includes the herringbone information for existing conditions only. The herringbone grate shall not be used for new construction.

At low velocities the vaned grate and herringbone grate are equally efficient. At higher velocities, greater than 5 ft/s (1.5 m/s), a portion of the flow tends to skip over the herringbone whereas the vaned grate will capture a greater portion of this flow. The vaned grate also has a higher capacity for passing debris and should be used for high debris areas.

The amount of flow bypassing the inlet on a continuous grade is computed as follows:

$$Q_{BP} = Q\left[\frac{(Z_D) - (GW)}{(Z_d)}\right]^{\frac{8}{3}}$$

Where: Q_{BP} = Portion of flow in cfs (cms) outside the width of the grate

Q = Total flow of gutter cfs (cms) approaching the inlet

Z_d = Top width of the flow prism in feet (meters)

GW = Gross width of the grate inlet in feet (meters) perpendicular to the direction of flow

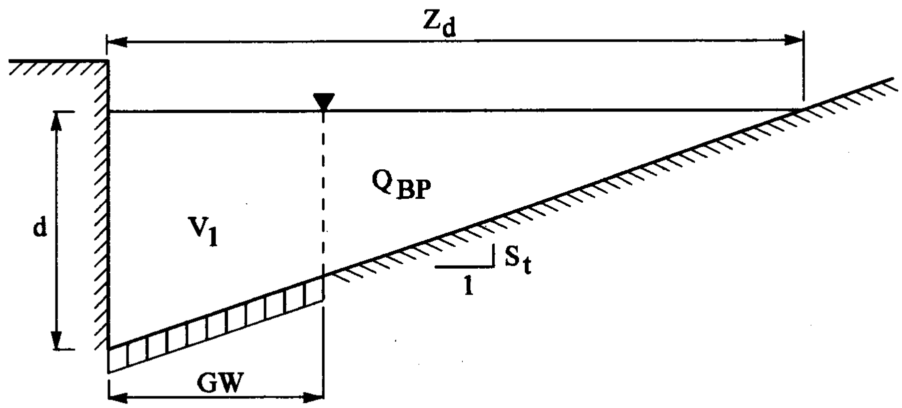
The velocity of the flow directly over the inlet is of interest to the designer. It may differ significantly from the average velocity. For an accurate analysis, this velocity should be between 3 and 5 ft/s (1.0 to and 1.8 m/s). When under 3 ft/s (1.0 m/s), the results of the analysis may be conservative (if side flow interception is ignored), see the first paragraph of this section for more guidance. The velocity is calculated as follows:

$$V_1 = \frac{Q - Q_{BP}}{(GW)\left[d - 0.5(GW)(S_t)\right]}$$

Where: S_t = Transverse slope or superelevation in ft/ft (m/m)

V_1 = Velocity over the inlet in ft/s (m/s)

d = Depth of flow at the face of the curb ft (m)



Section at Inlet
Figure 5-5.1

5-5.1.1 Example

The project is located in Seattle on a non-interstate roadway. The high point of a vertical curve is at Station 41+00. The width of pavement is 38 ft (11.5 m), with a 5 ft shoulder and 11 ft lanes. A proposed drainage system consists of grate inlets placed at the following stations:

Station	(S_L)Longitudinal Grade	(S_t) Superelevation
48+50	0.011	0.035
51+50	0.024	0.022
54+50	0.028	0.2
57+50	0.028	0.02
59+00	0.028	0.02

Complete a pavement and drain inlet analysis for this situation using the formulas below:

Solution:

Assume T_c=5 min for all inlets
Use 10-year MRI design rainfall

1. Determine the intensity, see section 2-4.4 for intensity equation and Figure 2-4.4B for m and n values.

$$I = \frac{m}{(T_c)^n} = \frac{5.62}{(5)^{0.530}} = 2.39\frac{in}{hr}$$

2. Determine the area of flow from the high point of the vertical curve (Station 41+00) to the first inlet (Station 41+70).

$$A = 38 ft \times ((48+50)-(41+00)) = 28500 ft^2$$

(Convert to acres; see Appendix A 1-1 for conversion.)

$$A = \frac{2660 ft^2}{43560 \frac{ft^2}{acre}} = 0.061 acres$$

3. Determine flow collected from Stations 41+00 to 41+70, see section 2-4 for equation.

$$\Delta Q = \frac{CIA}{K_c} = \frac{(0.9)(2.39)(0.65)}{1} = 1.41 cfs$$

4. The inlet at Station 48+50 is analyzed next. The depth of flow (d) and width of flow (Z_d) are calculated using the equations from Section 5-4. Verify Z_d is within the allowable limit: the shoulder width (5 ft) plus one-half of the traveled lane (5.50 ft) or 10.5 ft.

$$d = \left[\frac{\Delta Q S_t}{37(S_L)^{0.5}}\right]^{\frac{3}{8}} = \left[\frac{1.41 \times 0.035}{37(0.011)^{0.5}}\right]^{\frac{3}{8}} = 0.19 ft$$

$$Z_d = \frac{d}{S_t} = \frac{0.19}{.035} = 5.56 ft$$

Z_d is acceptable since Z_d = 5.56 ft which is less than the allowable limit of 10.5 ft. If Z_d is found to be quite small, a zero may be placed in the grate width column, which would indicate that no inlet is needed at this location. This may be checked repeatedly until Z_d approaches the allowable limit at which point an inlet should be located.

5. Q_{BP} is then calculated utilizing equations from Section 5-5.1, this is the portion of water that is flowing past the inlet and added to the flow for the next inlet (51+50).

$$Q_{BP} = \Delta Q\left[\frac{(Z_d)-(GW)}{Z_d}\right]^{\frac{8}{3}} = 1.41\left[\frac{(5.56)-(2.05)}{5.56}\right]^{\frac{8}{3}} = 0.41 cfs$$

6. Next, the amount of flow intercepted by the grate is determined.

$$Q_i = \Delta Q - Q_{BP} = 1.41 - 0.41 = 1.0 cfs$$

7. The velocity V_1 is checked to make sure it is acceptable (within 3 to 5 ft/s).

$$V_1 = \frac{\Delta Q - Q_{BP}}{(GW)[d-0.5(GW)(S_t)]} = \frac{1.41-0.41}{(2.05)[0.19-0.5(2.05)(0.035)]} = 3.16 \frac{ft}{s}$$

The velocity is within limit, so the designer can proceed to the next inlet. The velocity is not within limit, the designer might want to change the inlet spacing. If the velocity cannot practically be brought within limit, see the beginning of section 5-5.1 for more direction.

8. The designer then proceeds to the next inlet at Station 51+50 and repeats the analysis starting at numbers 2 and 3 as shown below:

$$A = 38ft \times ((51+50)-(48+50)) = 11400 ft^2$$

$$A = \frac{11400 ft^2}{43560 \frac{ft^2}{acre}} = 0.26 acres$$

$$\Delta Q = \frac{CIA}{K_c} = \frac{(0.9)(2.39)(0.26)}{1} = 0.56 cfs$$

At this point the by pass flow (Q_{BP}) from Station 48+50 needs to be added to delta flow (ΔQ)above to determine the total flow (ΣQ) to the inlet at Station 51+50.

$$\Sigma Q = Q_{BP} + \Delta Q = 0.41 + 0.56 = 0.97 cfs$$

The analysis continues at number 4 using the (ΣQ) above in place of (ΔQ).

The process is repeated as often as necessary until the last inlet is checked. The last inlet may require an adjustment of spacing (usually a smaller spacing) in order to comply with criteria to limit the last Q_{BP} to 0.10 cfs (0.003 m^3/s). Inlets should always have a minimum spacing of 20 ft (7 m) to enable the bypass water to reestablish its flow against the face of curb.

Keep in mind that the deeper a gutter flows, the more efficient the inlet will perform. The 300 ft (90 meter) spacing for inlets shown in Example 5-5.1.1 is not a requirement. This distance is a maintenance guideline for situations where catch basins dump directly into a trunk line.

5-5.2 Capacity of Inlets in Sag Locations

Inlets in sag locations perform differently than inlets on a continuous grade and therefore require a different design criterion. By definition, sag is any portion of the roadway where the profile changes from a negative grade to a positive grade. A sag location is of primary concern when water ponds more than half way into the nearest traveled lane due to all of the inlets being plugged with debris. This ponded area is generally contained by a curb, traffic barrier, retaining wall, or other obstruction, which prevents the runoff from flowing off of the traveled roadway.

A sag vertical curve is of less concern if the runoff is capable of overtopping the curb and flowing away from the roadway without ponding water over more than half of the nearest traveled lane. With this situation there is a low potential for danger to the drivers of the roadway if the inlets do not function as designed.

Theoretically, inlets that are placed in sag locations may operate in one of two ways. At low ponding depths, the inlet will operate as a weir; while at higher depths, the inlet will operate as an orifice. It is very rare that ponding on a roadway will be allowed to become deep enough to force the inlet to operate as an orifice. As a result, the inlet can be safely assumed to always operate as a weir with flow spilling in from the three sides of the inlet that are exposed to the ponding.

The capacity of an inlet operating as a weir is:

$$Q = C_W \times P \times d^{\frac{3}{2}}$$

To find the depth when Q is known, use the following equation:

$$d = \left[\frac{Q}{C_W P}\right]^{\frac{2}{3}}$$

Where: Q = Flow into the inlet in cubic feet per second, (cms)

C_w = Weir coefficient, 3.0 (1.66 for metric)

P = Effective grate perimeter, in feet (m), see Figure 5-5

d = Depth of ponded water at the inlet in feet (meters)

Any section of roadway located in a sag, as discussed in the first paragraph of this section, should be designed according to the following criteria:

1. One inlet should be placed at the lowest elevation in the sag. A combination inlet should be used at this location to provide continued inlet flow if the grate becomes plugged with debris. Use the gutter profile to determine this location instead of the centerline profile. The depth of water over this inlet is d_B.
2. Two additional flanking inlets should be placed on each side of the inlet described in the first criteria. These inlets can be regular grate inlets. The flanking inlets should be located so that the ponded water is $0.5d_B$ deep above the flanking inlets when half of the traveled lane is covered with ponded water.
3. The rainfall intensity for a 50-year MRI should be used for these three inlets.
4. The three inlets should be assumed to be 50 percent plugged (except for the Combination Inlet B-2e, which be considered 0% plugged), therefore the total available perimeter should be reduced by half in the analysis. This adjustment is in addition to reducing the perimeter to account for the obstruction caused by the bars in the grate. Figure 5-5 lists perimeters for various grates with reductions already made for bars.

5-5.2.1 Example

For this example, assume there is a roadway with a sag in its profile. Inlet spacing has already been calculated using the 10-year MRI for the continuous grade sections on either side of the sag.

The next step is to determine how much runoff will bypass the final inlet on either side of the sag using the 50-year MRI (see number 1 in example 5-5.1.1) This may create a higher than allowable width of flow at some inlet locations on the continuous grade section previously calculated; however, this is ignored since the flows are calculated only to determine how much flow will bypass the final inlet on the continuous grade and enter the sag during the 50-year MRI. For this example, use 0.1 cfs for a bypass flow from the down stationing side and 0.08 cfs for a bypass flow from the up stationing side (the bypass was found using number 5 of example 5-5.1.1).

1. The next step is to calculate the runoff, other than bypass flow, that is contributing to the ponding in the sag. This is done as described in example 5-5.1.1 and is the runoff that is generated from the pavement between the last inlet on either side of the continuous grades. It is calculated by determining the total pavement area downstream of the continuous grade inlets contributing runoff to the sag and applying the rational method using this area. The rational method is used in the same manner as when runoff is calculated for a continuous grade (see the example 5-5.1.1 numbers 2 and 3). For this example, use 0.72 cfs as the runoff from the pavement in the sag.

2. Once this flow value is calculated, it is added to the two bypass flows to determine the total flow contributing to the sag.

   $Q_{TOTAL} = Q_{BP1} + Q_{BP2} + Q_{SAG}$

   $Q_{TOTAL} = 0.1+0.08+0.72=0.90$ cfs

3. Next, $d_{B\ allowable}$ is checked. At the lowest point of the sag, in this example, the transverse slope or superelevation at the pavement edge is 0.02 ft/ft. Since the shoulder is 5 feet wide and the traveled lane is 11 feet wide, the allowable width of ponding (Z_d) is 10.5 feet (the shoulder width plus half of the traveled lane). The allowable depth of ponding at the sag is:

   $d_{B\ allowable} = S_T \times Z_d = 0.02 \times 10.5 = 0.21$ ft

4. The effective perimeter of the inlets is determined next. The three inlets must convey the total flow without causing more that 0.21 ft of ponding at the deepest point. The inlets will act as a weir with flow entering from three sides (the side against the curb neglected). If vaned grates are used for the inlets, the effective perimeter for each of the inlets is:

   0.5 (1.31+1.25+1.31) = 1.94 ft

   (The 0.5 factor accounts for 50 percent of the inlet being plugged.)

5. Next, determine the maximum allowable flow into all three inlets when maximum ponding occurs. The flow into the lowest inlet is calculated using the weir equation with the depth d_B. The flow into the two flanking inlets can be calculated using the weir equation with the depth $0.5d_B$. The weir equation can be set up to analyze all three inlets at once.

   $$\Sigma Q = C_W \times P \times \left[2(0.5d_B)^{\frac{3}{2}} + (d_B)^{\frac{3}{2}} \right]$$

   (See beginning of section 5-5.2 for definition of terms.)

   $$\Sigma Q = \left[3 \times 1.94 \times \left[2(.5 \times 0.21)^{\frac{3}{2}} + (0.21)^{\frac{3}{2}} \right] \right] = 0.95 cfs$$

   Since 0.95 > 0.93, the flow into the inlets at maximum allowable ponding is greater than the peak runoff contributing to the sag so the maximum allowable ponding will never be exceeded and the design is good. One inlet is placed at the lowest point of the sag and one flanking inlet is placed on each side of the lowest inlet at the station where the gutter elevation is $0.5d_B$ higher than the gutter elevation at the lowest point in the sag.

   If the flow into the inlets at maximum allowable ponding had been less than the peak runoff contributing, then the design would not be acceptable. The design would have to be performed again with additional flanking inlets included or with the three original inlets replaced with inlets that have larger openings. If additional flanking inlets are used, they should be placed close to the inlet at the lowest point in the sag to increase the flow into them. Also, if additional flanking inlets are used, the equation shown above will have to be modified to include the flow into the additional inlets.

6. The actual ponding depth at the sag can be determined by the following equation if the three inlet configuration is used.

$$d_{Ba} = \left[\frac{Q_{TOTAL}}{C_W \times P \times 1.707} \right]^{\frac{2}{3}} = \left[\frac{0.90}{3 \times 1.94 \times 1.707} \right]^{\frac{2}{3}} = 0.20\,ft$$

A worksheet of the steps outlined in this example can be found at the following web link http://www.wsdot.wa.gov/eesc/design/hydraulics/downloads.htm. Designers may find it useful to fill out the worksheet for each inlet located at a sag. Worksheets should be submitted with Hydraulics Reports.

5-6 Hydroplaning and Hydrodynamic Drag

As the depth of water flowing over a roadway surfaces increases, the potential for both hydroplaning and hydrodynamic drag increases. Both are discussed in more detail in the subsequent paragraphs below.

Hydrodynamic drag is a term used to describe the force applied to the tire of a vehicle pushing through water as opposed to the tire lifting off the pavement (hydroplaning). The differential force between the tire in the water and the tire out of the water causes the vehicle to "pull" or veer to the side of the water. This usually occurs at speeds less than 50 mph and in water deeper than the depth of the vehicles tire tread. Minimizing water flow depth across lanes and intrusion of flow into lanes will decrease the possibility of hydrodynamic drag.

When rolling tires encounter a film of water on the roadway, the water is channeled through the tire pattern and through the surface roughness of the pavement. Hydroplaning occurs when the drainage capacity of the tire tread pattern and the pavement surface is exceeded and the water begins to build up in front of the tire. As the water builds up, a water wedge is created and this wedge produces a hydrodynamic force, which can lift the tire off the pavement surface. This is considered as full dynamic hydroplaning and, since water offers little shear resistance, the tire loses its tractive ability and the driver loses of control of the vehicle.

Hydroplaning is a function of the water depth, roadway geometrics, vehicle speed, tread depth, tire inflation pressure, and conditions of the pavement surface. The following can reduce the hydroplaning potential of a roadway surface:

1. Design the highway geometries to reduce the drainage path lengths of the water flowing over the pavement. This will prevent flow build-up.
2. Increase the pavement surface texture depth by such methods as grooving of Portland cement concrete. An increase of pavement surface texture will increase the drainage capacity at the tire pavement interface.
3. The use of open graded asphaltic pavements has been shown to greatly reduce the hydroplaning potential of the roadway surface. This reduction is due to the ability of the water to be forced through the pavement under the tire. This releases any hydrodynamic pressures that are created and reduces the potential for the tires to hydroplane.
4. The use of drainage structures along the roadway to capture the flow of water over the pavement will reduce the thickness of the film of water and reduce the hydroplaning potential of the roadway surface.

6-1 Introduction

A storm drain is a network of pipes that convey surface drainage from catch basins or other surface inlets, through manholes, to an outfall. The network must have at least three pipes in any combination of laterals and trunks (see Figure 6-1) to be classified as a storm drain.

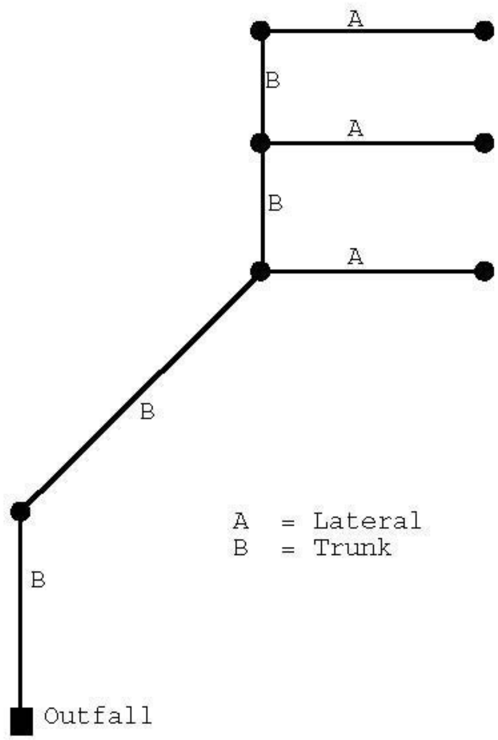
All storm drain designs will be based on an engineering analysis which takes into consideration runoff rates, pipe flow capacity, hydraulic grade line, soil characteristics, pipe strength, potential construction problems, and potential water quality issues. A majority of time spent on a storm drain design is used calculating runoff from an area and designing a pipe to carry the flow. A storm drain design may be performed by hand calculations or by one of several available computer programs and spreadsheets.

Runoff is calculated using either the rational method or the Santa Barbara Urban Hydrograph (SBUH) method, see Chapter 2 for further discussion. Pipe capacity is calculated using Manning's Equation, which relates the pipe capacity to the pipe diameter, slope, and roughness. The Region's Hydraulics Engineer reviews the design and if required the Headquarters (HQ) Hydraulics Office provides final approval as part of the hydraulics report review, see Chapter 1 for further approval guidelines.

6-2 Design Features

Along with determining the required pipe sizes for flow conveyance, storm drain system design incorporates the following features:

1. Soil Conditions — Soil with adequate bearing capacity must be present to interact with the pipes and support the load imparted by them. Surface and subsurface drainage must be provided to assure stable soil conditions. Soil resistivity and pH must also be known so the proper pipe material will be used.
2. Inlet Spacing and Capacity — Design guidelines are detailed in Chapter 5, Drainage of Highway Pavements.
3. Junction Spacing — Junctions (catch basins, grate inlets and manholes) should be placed at all breaks in grade and horizontal alignment. Pipe runs between junctions should not exceed 100 meters (300 feet) for pipes smaller than 1,200 millimeters (48 inches) in diameter and 150 meters (500 feet) for pipes 1,200 millimeters (48 inches) or larger in diameter. When grades are flat, pipes are small or there could be debris issues; designers should consider reducing the minimum spacing. Region Maintenance should be consulted for final approval on maximum spacing.
4. Future Expansion — If it is anticipated that a storm drain system may be expanded in the future, provision for the expansion shall be incorporated into the current design.
5. Velocity — Velocity of flow should be 3 feet per second (1.0 meter per second) or greater to prevent the pipes from clogging due to siltation. Velocity of flow should not be excessively high since high flow velocities (approaching and above 10 feet per second) produce very large energy losses in the storm drain system and also cause abrasion of the pipes. The velocity should be calculated under full flow condition even if the pipe is only flowing partially full with the design storm.



A lateral(s) discharges into a trunk line. The trunk line then receives the discharge and conveys it to an outfall.

Figure 6-1
Storm Drain Structure

6. Grades at Junctions — Pipe crowns of branch or trunk lines entering and exiting junctions should be at the same elevation. If a lateral is placed so its flow is directed against the main flow through the manhole or catch basin, the lateral invert must be raised to match the crown of the inlet pipe.
7. Minimum Pipe Diameter — The minimum pipe diameter shall be 300 millimeters (12 inches), except that single laterals less than 15 meters (50 feet) long may be 200 millimeters (8 inches) in diameter (some manufacturers are unable to add protective treatment for 200 mm storm drain pipe).
8. Energy Losses — Often energy losses do not need to be calculated (energy losses are calculated to determine the hydraulic grade line). However, in certain cases the losses occurring may be significant and will need to be calculated. Possible situations of concern include the following:
   - High flow velocities through the system.
   - The pipes are on flat slopes.
   - Inlet and outlet pipes forming a sharp angle at junctions.
   - Multiple flows entering a junction.
   - The pipes entering and leaving the junction are very shallow.

   Section 6-6 describes how to calculate energy losses.

9. Increase in Profile Grade — In cases where the roadway or ground profile grades increase downstream along a storm drain, a smaller diameter pipe may sometimes be sufficient to carry the flow at the steeper grade. However, due to maintenance concerns, the Washington State Department of Transportation (WSDOT) design practices do not allow pipe diameters to decrease in downstream runs.

   Consideration should be given in such cases to running the entire length of pipe at a grade steep enough to allow use of the smaller diameter pipe. Although this will necessitate deeper trenches, the trenches will be narrower for the smaller pipe and therefore the excavation may not substantially increase. A cost analysis is required to determine whether the savings in pipe costs will offset the cost of any extra structure excavation.

10. Outfalls — Outfalls must conform to the requirements of all federal, state, and local regulations. Storm runoff may need to receive water quality and quantity treatment before discharging from a storm drain system (see the WSDOT *Highway Runoff Manual*). Erosion is common and must be prevented at the storm drain outfall. Installation of tide gates may be necessary when the outfall is in a tidal area.

11. Location — Medians usually offer the most desirable storm drain location. In the absence of medians, a location beyond the edge of pavement on state right of way or on easements is preferable. It is generally recommended when a storm drain is placed beyond the edge of the pavement that a one trunk system, with connecting laterals, be used instead of running one trunk down each side of the road.

12. Confined Space and Structures - Per WAC 296, any structure (catch basin, manhole or grate inlet) more than 4’ in depth is considered a confined space. As such, any structure exceeding 4’ deep that could be accessed by personnel must be equipped with a ladder. Local maintenance should be consulted to determine if a Vactor Hose or personnel would access a structure. Structures should not exceed 15’ due to the limitations of WSDOT Vactor Trucks. Any design requiring a structure deeper than 15’ should consult HQ Hydraulics for design guidance.

6-3 Data for Hydraulics Report

The design of a storm drain system requires that data be collected and documented in an organized fashion. A Hydraulics Report should be submitted which contains the following items and related calculations (whether performed by hand or computer):

- Plan sheets showing: the location of the project and topographic contours at sufficient intervals to allow the drainage areas to be defined (usually 2 feet or 0.5 meters). Drainage areas must be extended to the outermost limits regardless of how far from the roadway this may be and should be outline or shaded. The proposed drainage system, points of flow interception, location of manholes, and location of outlets should be shown on this map and drainage patterns indicated by directional arrows.
- All calculations necessary for the design of the storm drain. These include calculations for runoff amount, gutter flow, inlet spacing, pipe sizes, and minor losses.
- Roadway profiles, cross-sections, and superelevations should be included to allow for the checking of flow patterns.
- A profile of the storm drain system showing invert elevations, manhole top and bottom elevations, existing utilities, and existing and finished ground line elevations.

6-4 Storm Drain Design — Handheld Calculator Method

6-4.1 General

Storm drain design can be accomplished with a handheld calculator using the rational method. Figure 6-4.1 may be used to show calculations that were performed using this method of analysis. Figure 6-4.1 has five divisions: location, discharge, drain design, drain profile, and remarks. These divisions are further expanded in the subsections below.

6-4.2 Location

The "Location" section gives all the layout information of the drain.

Column 1 gives a general location reference for the individual drain lines, normally by the name of a street or a survey line.

Columns 2 and 3 show the stationing and offset of the inlets, catch basins, or manholes either along a roadway survey line or along a drain line.

6-4.3 Discharge

The "Discharge" section presents the runoff information and total flow into the drain.

Column 4 is used to designate the drainage areas that contribute to particular point in the drain system. The drainage areas should be numbered or lettered according to some reference system on the drainage area maps. The type of ground cover (pavement, median, etc.) may be indicated. Since drainage areas must be subdivided according to soil and ground cover types, a drainage area may have several different parts.

Column 5 shows the area of the individual drainage areas listed in Column 4 in hectares (acres for English units).

Column 6 shows the rational method runoff coefficient (see Chapter 2). Each individual drainage area must have a corresponding runoff coefficient.

Column 7 is the product of Columns 5 and 6. Column 7 is also the effective impervious area for the subsection.

Column 8, the summation of CA, is the accumulation of all the effective impervious area contributing runoff to the point in the system designated in Column 2. All the individual areas in Column 7 contributing to a point in Column 2 are summed.

Column 9 shows the time of concentration to the structure indicated in Column 2. Section 2-4.3 of this manual details how to calculate the time of concentration. Generally the time chosen here would be the longest time required for water to travel from the most hydraulically remote part of the storm drain system to this point. This would include flow over the drainage basin and flow through the storm drain pipes. The time of concentration should be expressed to the nearest minute and never be less than 5 minutes.

When the runoff from a drainage area enters a storm drain and the time of concentration (Tc) of the new area is shorter than the accumulated Tc of the flow in the drain line, the added runoff should be calculated using both values for Tc. First the runoff from the new area is calculated for the shorter Tc. Next the combined flow is determined by calculating the runoff from the new area using the longer Tc and adding it to the flow already in the pipe. The Tc that produces the larger of the two flows is the one that should be used for downstream calculations for the storm drain line.

Washington State Department of Transportation

Storm Drain Design Calculations

Calculated By	Date	Checked By	Date	SR	Project	Sheet # Of Sheets

Location			Discharge										Drain Design				Drain Profile						
Drain Located On	From Station	To Station	Source Of Drainage	Drainage Area (A)	Runoff Coeff. (C)	CA	ΣCA	T_c To Col. 2 Station	Rainfall Intensity	Runoff	Con. Inflow	Total Flow	Pipe Diameter	Pipe Slope	Flow Velocity	Pipe Capacity	Length	Elev. Change	Upstr. Ground Elev.	Downstr. Ground Elev	Upstr. Invert Elev.	Downstr. Invert Elev.	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24

Storm Drain Design Calculations
Figure 6-4.1

The easiest method for determining the Tc of the flow already in the system (upstream of the structure in Column 2) is to add the Tc from Column 9 of the previous run of pipe (this value should be on the row above the row that is currently being filled in) to the time it took the flow to travel through the previous run of pipe. To determine the time of flow (or more correctly, the travel time) in a pipe, the velocity of flow in the pipe and the length of the pipe must be calculated. Velocity is computed using Manning's Equation and is found in Column 14 of the previous run of pipe. The length used is the value entered in Column 18 for the previous run of pipe. Obviously, this calculation is not performed for the very first (most upstream) run of pipe in a storm drain system.

$$T_1 = \frac{L}{60V}$$

where: Tt = Time of concentration of flow in pipe in minutes

L = Length of pipe in meters (feet)

V = Velocity in m/s (ft/s for English units)

The designer should note that this calculation assumes that the pipe is flowing full. It is accurate for pipes flowing slightly less than half full up to completely full. It will be slightly conservative for Tc calculations when the pipe is flowing significantly less than half full.

Column 10 shows the rainfall intensity corresponding to the time indicated in Column 9 and the location of the project.

The intensity is in millimeters per hour to the nearest tenth (inches per hour to the nearest hundredth for English units). The rainfall intensity used is a 10 year recurrence interval for laterals and a 25-year recurrence interval for trunk lines. See Chapter 2 for a complete description of how this intensity can be determined.

Column 11 shows the amount of runoff to the nearest hundredth of a cubic meter per second (nearest tenth of a cubic foot per second) up to the point indicated in Column 2. It is computed as the product of Columns 8 and 10. This is simply applying the rational method to compute runoff from all the drainage area upstream of the pipe being analyzed.

Column 12 shows any flow, other than the runoff calculated in Column 11, to the nearest hundredth of a cubic meter per second (nearest tenth of a cubic foot per second) that is entering the system up to the point indicated in Column 2. It is rare to have flow entering a system other than runoff from the drainage basin but this does occur. For instance, when an underdrain, which is draining ground water, is connected to the storm drain. The label for this column indicates that these flows are considered constant for the duration of the storm so they are independent of the time of concentration.

This column is also used when the junction is a drywell and a constant rate of flow is leaving the system through infiltration. When this occurs the value listed in Column 12 is negative. See Section 6-7 for a complete discussion of drywells.

Column 13 is the sum of columns 11 and 12 and shows the total flow in cubic meters per second to the nearest hundredth (cubic feet per second to the nearest tenth) to which the pipe must be designed.

6-4.4 Drain Design Section

This section presents the hydraulic parameters and calculations required to design storm drain pipes.

Column 14 shows the pipe diameter in millimeters (inches). This should be a minimum of 200 millimeters (8 inches) for any pipe with a length of 15 meters (50 feet) or less. Pipes longer than 15 meters (50 feet) must have a minimum diameter of 300 millimeters (12 inches). Pipe sizes should never decrease in the downstream direction.

The correct pipe size is determined through a trial and error process. The engineer selects a logical pipe size that meets the minimum diameter requirements and a slope that fits the general slope of the ground above the storm drain. The calculations in Column 17 are performed and checked against the value in Column 13. If Column 17 is greater than or equal to Column 13, the pipe size is adequate. If Column 17 is less than Column 13 the pipe does not have enough capacity and must have its diameter or slope increased after which Column 17 must be recalculated and checked against Column 13.

Column 15, the pipe slope, is expressed in meters per meter (feet per foot). This slope is normally determined by the general ground slope but does not have to match the surface ground slope. The designer should be aware of buried utilities and obstructions, which may conflict, with the placement of the storm drain.

Column 16 shows the full flow velocity. It is determined by Manning's Equation which is shown below. The velocity is calculated for full flow conditions even though the pipe is typically flowing only partially full. Partial flows will be very close to the full flow velocity for depths of flow between 30 percent and 100 percent of the pipe diameter.

$$V = \frac{1}{n}R^{2/3}\sqrt{S} = \frac{1}{n}\left[\frac{D}{4}\right]^{2/3}\sqrt{S} \qquad \text{(metric units)}$$

or

$$V = \frac{1.486}{n}R^{2/3}\sqrt{S} = \frac{1.486}{n}\left[\frac{D}{4}\right]^{2/3}\sqrt{S} \qquad \text{(English units)}$$

Where:
- V = Velocity in m/s (ft/s)
- D = Pipe diameter in meters (feet)
- S = Pipe slope in meters/meter (feet/foot)
- n = Manning's roughness coefficient (see Appendix 4-1)

Extremely high velocities should be avoided because of excessive abrasion in the pipe and erosion at the outlet of the system. Drop manholes should be considered for pipe velocities over 3.0 meters per second (10 feet per second). The engineer should also keep in mind that energy losses at junctions become significant above 2 meters per second (6 feet per second).

The minimum velocity as determined by this equation is 1 meter per second (3 feet per second).

Column 17, the pipe capacity, shows the amount of flow in cubic meters per second (cubic feet per second) which can be taken by the pipe when flowing full. It is computed using the following formula:

$$Q = VA = V\frac{\pi D^2}{4}$$

Where: Q = Full flow capacity in cfs (cms)

V = Velocity as determined in Column 16 in ft/s (m/s)

A = Cross sectional area of pipe in feet squared (meters sq)

D = Diameter of pipe in feet (meters)

6-4.5 Drain Profile

The drain profile section, Columns 18 through 23, includes a description of the profile information for each pipe in the storm drain system. It describes the pipe profile and the ground profile. The ground elevations should be finished elevations. The items in this section are generally self-explanatory. The only exception is Column 18, the length shown is the horizontal projection of the pipe, in meters (feet), from centers of appurtenances. Generally, profiles should be set to provide a minimum of 0.6 meters (2 feet) of cover over the top of the pipe.

6-4.6 Remarks

Column 24 Remarks is for any information, which might be helpful in reviewing the calculations. This space should note unique features such as drop manholes, long times of concentration, changes in the type of pipe, or changes in design frequency.

6-5 Storm Drain Design — Computer Analysis

In recent years, with the addition of personal computers to most engineering work stations, storm drain design by handheld calculator has become less prevalent. Storm drain design by computer analysis offers some distinct advantages over calculations performed by hand. Chief among these advantages is the decreased amount of time required to perform the pipe sizing and hydraulic grade line calculations and the reduced chance for calculation errors.

Some computer programs will use the rational method for storm drain design while others will use a hydrograph method such as the SBUH method. Both of these methods are valid for WSDOT storm drain design; however, they will yield different peak runoff values. This is most distinct for drainage basins that have very short times of concentration. As a basin's time of concentration extends beyond 15 minutes the two methods yield more similar answers. This difference in peak runoff values ends up having little effect on storm drain design since runoff from basins with short times of concentration tends to be small and the required pipe size is determined by the minimum allowable pipe size. As flows entering the system increase to the point that minimum pipe sizes are no longer the governing factor, the associated time of concentration becomes greater and the two methods produce similar peak flow rates.

There are several commercially available computer programs for storm drain design. Each of these programs have certain features that make them unique from other programs but the primary calculations are performed the same way. Because of this, nearly any commercially available computer program that performs storm drain design is acceptable for designing WSDOT storm drains.

The Washington State Ferries Division and each WSDOT Region has available for designers use the computer program Storm Shed. The HQ Hydraulics Branch encourages the use of Storm Shed whenever designing a storm drains and is available to lend technical assistance. Prior to using Storm Shed, a Microsoft® Excel Pavement Drainage Spreadsheet should be used to locate each storm drain. A basic layout is shown in Figure 6-4.1 and the spreadsheet is available on the HQ Hydraulic web page at: http://www.wsdot.wa.gov/eesc/design/hydraulics. The spreadsheet lacks the advanced features found in commercially available computer programs but does offer a simple and effective way to locate storm drains.

6-6 Hydraulic Grade Line

The hydraulic grade line (HGL) represents the water surface elevation of the flow traveling through the storm drain system. If the HGL becomes higher at a manhole or catch basin than the rim elevation of that structure, flow will leave the storm drain. This can cause severe traffic safety problems and must always be avoided. Fortunately, if the storm drain pipes were designed as discussed in the previous sections, then the HGL will only become higher than the catch basin or manhole rims when energy losses become significant or if the storm drain is on a very flat gradient. However, the HGL should always be evaluated especially when energy losses become significant or when the pipes are installed at very flat gradients.

Typically when flow velocities in storm drains are moderate (less than 2 m/s), energy losses are insignificant and can be ignored. When flow velocities become higher then energy losses need to be calculated. Once energy losses are calculated, the HGL can be calculated to determine if the storm drain will function properly.

The HGL can only be calculated after the storm drain system has been designed. The HGL is calculated beginning at the most downstream point of the storm drain and ending at the most upstream point, which is exactly the opposite direction that was used to design the pipe sizes. The water surface elevation at the storm drain outfall must be known or calculated since it acts as the starting elevation of the HGL. Refer to Chapter 3 for an explanation on calculating water surface elevations at the downstream end of a pipe (the tailwater is calculated the same for storm drain outfalls and culverts). Once the tailwater elevation is known, the energy loss (usually called head loss) from friction is calculated for the most downstream run of pipe and the applicable minor losses are calculated for the first junction upstream of the outfall. All of these head losses are added to the water surface elevation at the outfall to obtain the water surface elevation at the first upstream junction (also the HGL at that junction). The head losses are then calculated for the next upstream run of pipe and junction and they are added to the water surface elevation of the first junction to obtain the water surface elevation of the second upstream junction. This process is repeated until the HGL has been computed for each junction. The flow in most storm drain pipes is subcritical; however, if any pipe is flowing supercritical (see Chapter 3 for an explanation of subcritical and supercritical flow) the HGL calculations are restarted at the junction on the upstream end of the pipe flowing supercritical. The HGL calculation process is represented in equation form below:

$$WSEL_{J1} = WSEL_{OUTFALL} + H_{f1} + H_{e1} + H_{ex1} + H_{b1} + H_{m1}$$
$$WSEL_{J2} = WSEL_{J2} + H_{f2} + H_{e2} + H_{ex2} + H_{b2} + H_{m2}$$
$$\ldots$$
$$WSEL_{Jn+1} = WSEl_{Jn} + H_{fn} + H_{en} + H_{exn} + H_{bn} + H_{mn}$$

where: WSEL = Water surface elevation at junction noted

H_f = Friction loss in pipe noted (see Section 6-6.1)

H_e = Entrance head loss at junction noted (see Section 6-6.2)

H_{ex} = Exit head loss at junction noted (see Section 6-6.2)

H_b = Bend head loss at junction noted (see Section 6-6.3)

H_m = Multiple flow head loss at junction noted (see Section 6-6.4)

As long as the HGL is lower than the rim elevation of the manhole or catch basin, the design is good. If the HGL is higher than the rim elevation the design is not acceptable since this indicates that flow will be exiting the storm drain system at that particular junction. The most common way to lower the HGL below the rim elevation is to lower the pipe inverts for one or more runs of the storm drain.

6-6.1 Friction Losses in Pipes

Head loss due to friction is a result of the kinetic energy lost as the flow passes through the pipe. The rougher the pipe surface is, the greater the head loss is going to be. Head loss from friction can be calculated with the following equation.

$$H_f = L\left[\frac{3.19\,Qn}{D^{2.667}}\right]^2 \text{ (metric units)}$$

$$H_f = L\left[\frac{2.15\,Qn}{D^{2.667}}\right]^2 \text{ (English units)}$$

where: H_f = Head loss due to friction in meters (feet for English units)

L = Length of pipe run in meters (feet)

Q = Flow in pipe in m^3/s (cfs)

6-6.2 Junction Entrance and Exit Losses

When flow enters a junction, it loses all of its velocity. As a result, there is an associated head loss equal to one velocity head. Then when the flow exits the junction and accelerates into the next pipe, there is another head loss equal to approximately half of one velocity head. These two head losses can be represented with the following equations (metric and English units use the same equations).

$$H_e = \frac{V_2^2}{2g}$$

$$H_{ex} = 1.0\left(\frac{V^2}{2g} - \frac{V_d^2}{2g}\right) \approx \frac{V^2}{4g}$$

Where H_e = head loss from junction entrance in meters (feet in English units)

H_{ex} = head loss from junction exit in meters (feet)

V = flow velocity in pipe in meters per second (feet per second)

V_d = channel velocity downstream of outlet in meters per second (feet per second)

g = gravitational acceleration constant

6-6.3 Losses From Changes in Direction of Flow

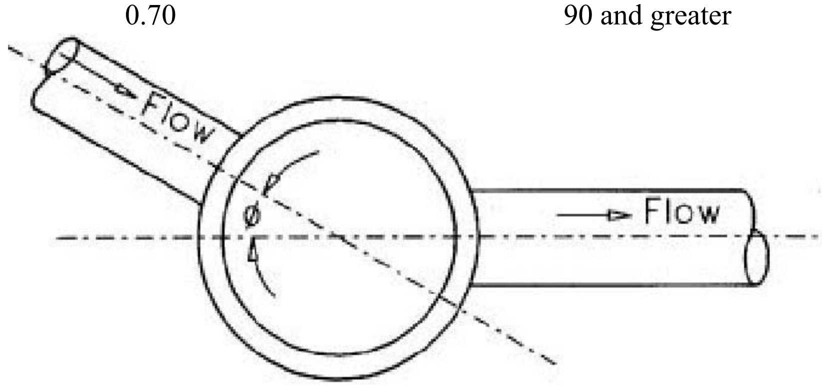
When flow changes direction inside of a junction, there is an associated head loss. The amount of head loss that will occur is dependent on how great the change is. As the angle between the inflow and outflow pipes increase, the amount of head loss increases. This head loss can be calculated with the following equation (metric and English units use the same equation).

$$H_b = K_b \frac{V^2}{2g}$$

where: H_b = Head loss from change in direction in meters (feet in English units)

K_b = Head loss coefficient for change in direction, see below:

K_b	Angle of Change in Degrees
0.00	0
0.19	15
0.35	30
0.47	45
0.56	60
0.64	75
0.70	90 and greater



Changes in Direction of Flow

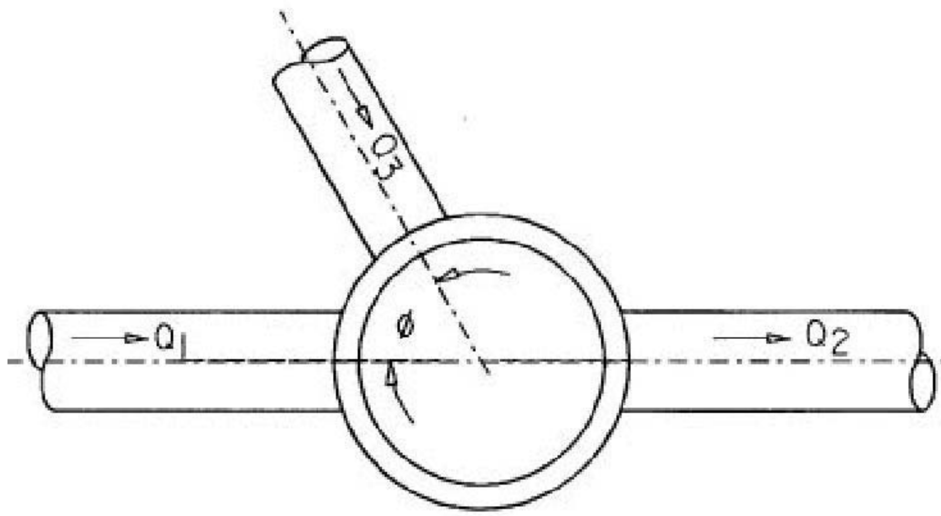
Figure 6-6.3

6-6.4 Losses From Multiple Entering Flows

When flow enters a junction from more than one pipe there is an associated head loss. The head loss is dependent on the amount of flow in each pipe and direction that each pipe enters the junction. This head loss can be calculated with the following equation (metric and English units use the same equation).

$$H_m = \frac{Q_2 V_2^2 - Q_1 V_1^2 - \cos\phi Q_3 V_3^2}{2gQ_2}$$

Where H_m = Head loss from multiple flows in meters (feet in English units)



Multiple Flows Entering a Junction
Figure 6-6.4

6-7 Drywells

A drywell is a manhole or catch basin that is perforated to allow flow in the storm drain to exit the system. Standard Plan B-27 of the WSDOT Standard Plans for Road, Bridge, and Municipal Construction depicts a typical drywell.

When soil conditions are appropriate, the use of drywells in a storm drain system can be beneficial. The primary advantage of drywells is that they reduce the flow in a storm drain system and thus reduce the sizes of the pipes in the system. Some highly effective drywells can completely eliminate the need for storm drain pipes. A secondary advantage of drywells is that they reduce flooding by discharging flow into ground water instead of discharging it to surface waters such as rivers and creeks.

For installation of a drywell to be practical, the surrounding soil should have an infiltration rate greater than or equal to 500 millimeters per hour (20 inches per hour). If the soil infiltration rate is less than this amount, the flow leaving the storm drain system will not be significant enough to justify the extra cost of installing a drywell. Soil infiltration rates should always be determined by a licensed geotechnical engineer soil analysis. Designing with incorrect soil infiltration rates is a primary cause of failure for storm drain systems with drywells.

Drywells can be designed with inlet and outlets pipes or without pipes. In either case, the designer must first determine the maximum amount of flow that will leave the system through the drywell. To accomplish this, the designer must determine the area of soil that will be wetted and allowing infiltration when the drywell is filled to its maximum allowable design elevation. This area is then multiplied by the infiltration rate of the soil, and usually also by a units conversion factor, to determine the flow rate out of the drywell in cubic meters per second (cubic feet per second). The flow rate is then used in one of two ways.

If the drywells are part of a storm drain system, which is connected by pipes, the calculated rate of infiltration flow leaving the drywell is subtracted from the flow entering the junction represented by the drywell. When using Figure 6-4.1 to perform the calculations, the flow infiltrating out of the drywell should be shown in Column 14 as a negative value to indicate that there is a constant flow leaving the system at this point. The designer should note that normally pipe sizes are not allowed to decrease in the downstream direction; however, if the flow value in Column 15 becomes less than zero, there may be no need for an outlet pipe since all flow will leave the system through drywell infiltration.

If the drywells are standing alone, that is there are no pipes connecting them and the only flow into them is through a grate on top of each drywell, the design is performed by simply calculating the amount of flow that enters the drywell through the grate and comparing it with the peak rate of flow that will infiltrate from the drywell. The designer must limit the amount of area draining to each drywell such that the flow out of the drywell through infiltration always exceeds the amount of flow entering the drywell.

Designers should be aware of potential impacts drywell infiltration may have on ground water. Removing pollutants from runoff, also referred to as water quality treatment, before discharging to ground water is always advisable. In many areas of Western Washington and some areas of Eastern Washington, water quality treatment is required prior to infiltrating runoff. In the other areas of the state, water quality treatment is left to the designer's discretion. See WSDOT's *Highway Runoff Manual* for a complete discussion on water quality treatment.

6-8 Construction Materials and Practices for Drains

Construction shall be performed in accordance with Section 7-04 of the WSDOT Standard Specifications for Road, Bridge, and Municipal Construction (Standard Specifications).

6-8.1 Structural Design

Most storm drains are placed at fairly moderate trench depths so the strength of the pipe is generally not a problem. Standard fill height tables are listed in Chapter 8 and will show the correct pipe for most designs. If the depth of cover is shallow (less than 0.6 meters) and truck wheel loads are present, it will be necessary to perform a special design to prevent structural damage to the pipe. Extreme fill heights may also cause structural damage to pipes and will thus require a special design. The HQ Hydraulics Branch shall be consulted whenever storm drain pipes are installed outside of the limits listed in the fill height tables.

The strength of the pipe depends upon its bed design, backfilling methods, and the quality of the backfill soil. In material incapable of developing adequate support, excavation and backfilling with granular material must be performed to a width sufficient to provide the necessary support.

6-8.2 Pipe Materials for Storm Drains

There are various pipe materials that are acceptable to WSDOT for storm drains. Chapter 8 in this manual and Section 7-04.2 and Section 9-05 of the Standard Specifications, describe the acceptable alternates. WSDOT's policy is to allow and encourage all possible alternates that will ensure a properly functioning storm drain at a reasonable cost. If at any specific location one or more of the alternates are not satisfactory, the unacceptable alternate or alternates shall be so stated on the plans usually in the structure note sheet. Storm drain pipe is subject to some use restrictions which are detailed in Chapter 8.

Pipe flow capacity depends on the roughness coefficient, which is a function of pipe material and manufacturing method. Fortunately, most storm drain pipes are 600 millimeters (24 inches) in diameter or less. Studies have shown that all the common pipe materials have a similar roughness coefficient in these sizes. For calculations, the designer should use a roughness coefficient of 0.013 for all pipes 600 millimeters (24 inches) or smaller. For larger diameter pipes, the designer should calculate the required pipe size twice: once for smooth wall pipe and again for corrugated pipe. If there is a difference in size, it should be indicated on the structure note sheet.

In estimating the quantity of structure excavation for design purposes at any location where alternate pipes are involved, estimate the quantity of structure excavation on the basis of concrete pipe since it has the largest outside diameter.

6-8.3 Pipe Joints for Storm Drains

Rubber gasketed joints are required for all drain pipe installations as per Section 7-04.3(2)E and Section 9-05 of the Standard Specifications.

6-8.4 Testing Storm Drains

Storm drains are required to be tested by one of the methods described in Section 7-04.3(4) of the Standard Specifications.

6-9 Subsurface Drainage

Subsurface drainage is provided for control of ground water encountered at highway locations. Ground water, as distinguished from capillary water, is free water occurring in a zone of saturation below the ground surface. The subsurface discharge depends on the effective hydraulic head and on the permeability, depth, slope, thickness, and extent of the aquifer.

The solution of subsurface drainage problems often calls for specialized knowledge of geology and the application of soil mechanics. The designer should work directly with the Region Materials Engineer as subsurface conditions are determined and recommendations are made for design in the Soil's Report.

Typical subdrain installations would be those provided for control of seepage in cuts or side hills or the lowering of the ground water table for proper subgrade drainage.

Subsurface drainage pipe size is determined by the same method used to design regular storm drain pipes. The only difference is that the flow used for the calculations is the predicted infiltration from groundwater into the system instead of flow entering the system from roadway drainage. When subsurface drainage is connected to a storm drain system, the invert of the underdrain pipe shall be placed above the operating water level in the storm drain. This is to prevent flooding of the underdrain system and defeating its purpose.

7-1 Introduction

Most rivers and creeks in Washington State contain one or more species of fish during all or part of the year. These fish must be allowed to freely migrate up and down the stream they occupy. If roadways are constructed across the stream without thought given to fish passage, the roadway can create a migration barrier. However, a correctly designed stream crossing will not hinder migration of fish.

The Washington State Department of Fish and Wildlife (WDFW) developed guidelines for any permanent road crossing culverts to facilitate upstream fish migration titled 'Design of Road Culvert for Fish Passage'. It provides direction for projects involving new culvert construction as well as retrofitting or replacing existing culverts.

At printing time, this chapter was still under development. A future revision is being drafted for the September 2004 *Hydraulics Manual* and will contain the following elements:

1. A summary of WDFW approaches for ensuring fish passage.
2. Direction for how to work with both WSDOT's and WDFW's requirements.
3. A discussion of structure types and recommended placement for fish passage.

For now, this chapter provides a brief summary of WDFW design approaches and notes the type of structures recommended for fish passage. For more guidance, designers should consult Region Hydraulics and the WDFW guideline by visiting http://www.wa.gov/wdfw/hab/engineer/cm/.

The basic concept used to ensure continued fish passage is to design the stream crossing to match the natural river or creek channel as much as practical. The idea being that if fish migration occurs in the stream before construction of the crossing, then migration should continue to occur after construction if the crossing (in other words flow conditions that are similar to natural flow conditions). For some types of crossing structures, it is easy to create flow conditions that are exactly like the natural flow conditions, but for other types of crossing structures, a detailed analysis is necessary to accomplish an acceptable design.

The first step in designing for fish passage is to determine which, if any, species of fish are in the stream that is being crossed by the road. WSDOT regional environmental staff is the primary contact for this information. They will contact the WDFW when necessary. Knowing the species of fish that will need to be designed for is important because the swimming and leaping ability of fish varies from species to species, so the design criteria will also vary. Knowing the species of fish present in the stream is also important because different species migrate through the stream during different times of the year, and as a result, the design flow that is used for the analysis must correlate with the time of year that the fish are migrating.

7-2 Types of Structures

7-2.1 General

For fish passage purposes, there are three basic types of stream crossing structures:

1. **Bridges**— Structures that have piers or abutments supporting some type of girder system.
2. **Open Bottom Culverts**— Metal and concrete arches or three sided concrete frame structures that have no floor and are supported by footings.
3. **Full Culverts** — Metal, concrete, and plastic round, pipe arch, elliptical, and box-shaped culverts that are completely enclosed self supporting structures.

7-2.3 Culvert Design Approach

Adequate fish passage at a culvert can be determined using 3 different design options:

1. No-Slope Design Option – Results in reasonably sized culverts without requiring much in the way of calculations. It is most effective for relatively short culverts at low-gradient sites. Culverts are typically larger than the hydraulic option, however the design avoids the additional cost of surveying and engineering. This is WDFW preferred method whenever possible.
2. The Hydraulic Design Option – Requires hydrologic and open-channel hydraulic calculations and specific site data, but usually results in smaller culverts that the no slope option. The analysis is based on velocity, depth and maximum turbulence requirements for a target species and age class.
3. Stream Simulation Design Option – Results in an artificial stream channel is constructed inside the culvert, thereby providing passage for any fish that would be migrating through the reach.

Chapter 8 Pipe Materials

8-1 Classification of Pipe

The Washington State Department of Transportation (WSDOT) utilizes a number of different types of pipe for highway construction activities. In order to simplify contract plan and specification preparation, pipes have been grouped into categories, and each category is intended to serve specific purposes.

When AASHTO or ASTM standards are referenced, the current year standards shall apply.

8-1.1 Drain Pipe

Drain pipe is small diameter pipe (usually less than 24 inch (600 mm)) used to convey roadway runoff or groundwater away from the roadway profile. Drain pipe is not allowed to cross under the roadway profile, and the minimum design life expectancy is 25 years. No protective treatment is required. Drain pipe is intended to be used in locations that can be accessed easily should it become necessary to maintain or replace the pipe.

Typical drain pipe applications include simple slope drains and small diameter "tight lines" used to connect underdrain pipe to storm sewers. Slope drains generally consist of one or two inlets with a pipe conveying roadway runoff down a fill slope. These drain pipes are relatively easy to install and are often replaced when roadway widening or embankment slope re-grading occurs. Slope drains are generally most critical during the first few years after installation, until the slope embankment and vegetation have had a chance to stabilize.

Drain pipe smaller than 12 inch (300 mm) can withstand fill heights of 30 feet (10 meters) or more without experiencing structural failure. All of the materials listed in Division 7-01 of WSDOT's *Standard Specifications for Road, Bridge, and Municipal Construction* (*Standard Specifications*) are adequate under these conditions. For drain pipe applications utilizing pipe diameters 12 inch (300 mm) or larger, or with fill heights greater than 30 feet (10 meters), the designer should specify only those materials that are listed in both Division 7-01 of the *Standard Specifications* and the fill height tables of Section 8-11.

The ends of thermoplastic drain pipe have a tendency to become buoyant and float. A length of metal or concrete pipe should be specified for the outlet when the drain pipe discharges to daylight. A detail for this design can be obtained from the Regional Hydraulic Section/Contact or the Olympia Service Center (HQ) Hydraulics Branch

8-1.2 Underdrain Pipe

Underdrain pipe is small diameter perforated pipe intended to intercept groundwater and convey it away from areas such as roadbeds or from behind retaining walls. Typical underdrain applications utilize 6 to 8 inch (150 to 200 mm) diameter pipe, but larger diameters can be specified. The minimum design life expectancy is 25 years, and no protective treatment is required.

Underdrain pipe is generally used in conjunction with well-draining backfill material and a construction geotextile. Details regarding the various applications of underdrain pipe are described in WSDOT *Design Manual* Chapter 530 and WSDOT CADD Detail Library.

8-1.3 Culvert Pipe

A culvert is a conduit under a roadway or embankment used to maintain flow from a natural channel or drainage ditch. Culverts are generally more difficult to replace than drain pipe, especially when located under high fills or major highways. Because of this, a minimum design life expectancy of 50 years is required for all culverts. Metal culvert pipes require a protective asphalt coating at some locations. Details are described in Section 8-5.3.1.

In order to determine the maximum and minimum amounts of cover that can be placed over a pipe, a structural analysis has been performed on the most commonly used pipe materials accepted by WSDOT for culvert applications. The results of these analyses are shown in the fill height tables of Section 8-11. For materials or sizes not shown in the tables of Section 8-11, contact the HQ Hydraulics Branch or section 7-02 of the *Standard Specifications*.

The design of culverts is discussed in Chapter 3. In addition to the hydraulic constraints of a location, the final decision regarding the appropriate culvert size to be used may be governed by fish passage requirements as discussed in Chapter 7.

Culvert shapes, sizes, and applications can vary substantially from one location to another. Listed below is a discussion of the various types of culverts that may appear on a typical contract.

8-1.3.1 Circular and Schedule Culvert Pipe

Circular culvert pipe from 12 inch (300 mm) to 48 inch (1200 mm) in diameter is designated as schedule pipe. The pipe schedule table is listed in Division 7-02 of the *Standard Specifications*. The pipe schedule table lists all of the structurally suitable pipe alternates available for a given culvert diameter and fill height. Schedule A culvert pipe is for fill depths from 2 feet to 15 feet (0.6 m to 4 m). Schedules B, C, and D pipe are for progressively higher fills. The contractor has the option of furnishing any of the pipe alternates that are listed on the schedule table. All schedule pipe shall be installed in accordance with Standard Plan B-11.

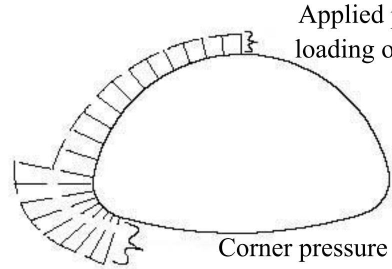
Schedule culvert pipe should be specified as "Schedule _____ Culv. Pipe ____ in (mm) Diam." on the contract plan sheets. Schedule pipe must be treated with the same protective coatings as other culvert pipe. The designer must insure that all of the alternates listed in the schedule table are appropriate for use at a given site and that the proper protective treatment is specified on the structure note sheets. If a particular type of schedule pipe is not appropriate for use, a structure note must be included that removes that culvert type from the schedule table.

The type of material for circular culvert pipe from 54 inch (1350 mm) to 120 inch (3000 mm) shall be designated on the plan sheets. The structure notes sheet should include any acceptable alternate material for that particular installation. A schedule table for these large sizes has not been developed due to their limited use. Also, structural, hydraulic, or aesthetic issues may control the type of material to be used at a site, and a specific design for each type of material available is generally necessary.

8-1.3.2 Pipe Arches

Pipe arches, sometimes referred to as "squash pipe," are circular culverts that have been reshaped into a structure that has a circular top and a relatively flat, wide bottom. For a given vertical dimension, pipe arches provide a larger hydraulic opening than a circular pipe. This can be useful in situations with minimal vertical clearances. Pipe arches also tend to be more effective than circular pipe in low flow conditions (such as fish passage flows) because pipe arches provide a majority of their hydraulic opening near the bottom of the structure, resulting in lower velocities and more of the main channel being spanned.

The primary disadvantage to using pipe arches is that the fill height range is somewhat limited. Due to the shape of the structure, significant corner pressures are developed in the haunch area as shown in Figure 8-1.3.2. The ability of the backfill to withstand the corner pressure near the haunches tends to be the limiting factor in pipe arch design and is demonstrated in the fill height tables shown in Section 8-11.



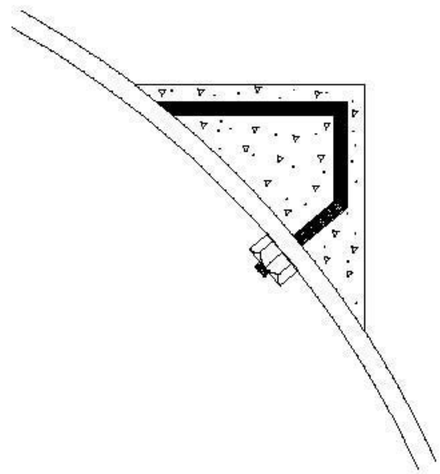
Typical Soil Pressure Surrounding a Pipe Arch
Figure 8-1-3.2

8-1.3.3 Structural Plate Culverts

Structural plate culverts are steel or aluminum structures that are delivered to the project site as unassembled plates of material and are then bolted together. Structural plate culverts are typically large diameter (from 10 feet (3 meters) to 40 feet (12 meters) or more) and are available in a number of different shapes including circular, pipe-arch, elliptical, and bottomless arch with footings. These structures are generally designed to span the main channel of a stream and are a viable option when fish passage is a concern.

The material requirements for structural plate culverts are described in Division 7-03 of the *Standard Specifications*. Aluminum structural plate culverts can be used anywhere in the state, regardless of the corrosion zone. Steel structure plate culverts are not permitted in salt water or Corrosion Zone III, as described in Section 8-4. The protective coatings described in Section 8-5.3.1 should not be specified for use on these types of culverts because the asphalt coatings interfere with the bolted seam process. In order to compensate for the lack of protective treatment, structural plate furnished in galvanized steel shall be specified with 1.5 oz/ft^2 (460 g/m^2) of galvanized coating on each surface of the plate (typical galvanized culvert pipe is manufactured with 1 oz/ft^2 (305 g/m^2) of galvanized coating on each surface of the pipe). The designer of structural plate culverts may also add extra plate thickness to the bottom plates to compensate for corrosion and abrasion in high-risk areas. Increasing the gauge thickness in this manner can provide a service life of 50 years or more for a very small increase in cost.

To prevent excessive deflection due to dead and/or live loads on larger structural plate culverts, longitudinal or circumferential stiffeners are sometimes added. Circumferential stiffeners are usually metal ribs bolted to the outside of the culvert. Longitudinal stiffeners may be metal or reinforced concrete thrust beams, as shown in Figure 8-1.3.3. The thrust beams are added to the structure prior to backfill. Concrete thrust beams provide some circumferential stiffening as well as longitudinal stiffening. They also provide a solid vertical surface for soil pressures to act on and a surface, which is easier to backfill against.



Concrete Thrust Beams Used as Longitudinal Stiffeners
Figure 8-1.3.3

Another method that can be used to diminish the loads placed on large span culverts is to construct a reinforced concrete distribution slab over the top of the backfill above the culvert. The distribution slab is generally used in low-cover applications and serves to distribute live loads out into the soil column adjacent to the culvert. The HQ Hydraulics Branch should be consulted to assist in the design of this type of structure.

8-1.3.4 Private Road Approach and Driveway Culverts

The requirements for culverts placed under private road approaches and driveways are less stringent than the requirements for culverts placed under roadways. Private road approach and driveway culverts are off of the main line of the highway, so very little hazard is presented to the traveling public if a failure occurs. Also, in many instances it is difficult to provide a minimum of 2 feet (0.6 m) of cover over the top of these culverts. Therefore, private road approach and driveway culverts can be specified without the protective treatments described in Section 8-5.3.1, and the minimum fill heights listed in Section 8-11 can be reduced to 1 foot (0.3 m). If fill heights less than 1 foot (0.3 m) are expected, concrete pipe of the class described in Fill Height Table 8-11.2 should be specified.

The designer is cautioned that structural failure may occur on some private road approaches or driveways if the right combination of fill height, live load, soil conditions, and pipe material are present. If live loads approaching an AASHTO HS-25 loading will consistently be traveling over the culvert and if the fill height is less than 2 feet (0.6 m), it is highly recommended that only concrete pipe of the class described in Fill Height table 8-11.2 be specified.

8-1.3.5 Concrete Box Culverts

Concrete box culverts are either cast-in-place or precast. All precast box culverts shall be installed in accordance with the manufacturer's recommendations. For extending or new construction of cast-in-place box culverts, please contact HQ Hydraulics. The dimensions and reinforcement requirements for precast box culverts are described in one of two specifications produced by the Association of State Highway and Transportation Officials (AASHTO). AASHTO M259 describes precast box culverts with fill heights ranging from 2 feet to 20 feet (0.6 to 6 meters). AASHTO M259 describes precast box culverts with fill heights less than 2 feet (0.6 m). See Section 8-10.7 for additional guidance on the use of concrete structures in shallow cover applications. If a precast box culvert is specified on a contract, the appropriate AASHTO specification should be referenced, along with a statement requiring the contractor to submit engineering calculations demonstrating that the box culvert meets the particular requirements of the AASHTO specification.

8-1.3.6 Concrete Three-sided Box Culverts

Concrete three-sided structures refer to either rectangular or arch shaped structures that are precast with reinforced concrete. The structures are generally supported by concrete footings, but can be fabricated with a full floor section if necessary. When footings are used, the footing slope should not be greater than 4% in the direction parallel to the channel. The structures are well suited for low cover applications where a relatively wide hydraulic opening must be provided. They can be specified with as little as zero cover and span lengths up to 26 feet (8 meters). It is possible to utilize structures with greater span lengths, but the design for those structures must be coordinated with the Bridge and Structures Office. The structures can be installed very quickly, often within one to two days, which can significantly decrease road closures or traffic delays. In addition to the hydraulic opening required, a location must be evaluated for suitability of the foundation material, footing type and size, and scour potential. The HQ Hydraulics Branch should be contacted to perform the necessary analysis and to provide cost estimates for PS&E purposes.

8-1.4 Storm Sewer Pipe

A storm sewer is defined as three or more pipes connected by inlets, catch basins, or manholes for the purpose of collecting pavement drainage. Storm sewers are usually placed under pavement in urbanized areas and for this reason are very costly to replace. The minimum design life of a storm sewer pipe is 50 years. All storm sewer pipes, unless indicated otherwise on the plans, must be pressure tested. Pressure testing is required primarily to indicate the presence of leaking seams or joints or other structural failures that may have occurred during the manufacturing or installation of the pipe. Division 7-04 of the *Standard Specifications* describes three types of pressure tests that are available. The contractor generally has the option of choosing which pressure test to perform. The tests include:

Exfiltration: The section of pipe to be tested is filled with water, and an apparatus is connected to the upper end of the pipe so that an additional 6 feet (2 m) of water column is placed on the test section. The leakage out of the pipe is measured, and must be less than the allowable leakage described in the *Standard Specifications.*

Infiltration: This test is intended for situations where the groundwater table is above the crown of the upper end of the pipe test section. Once the pipe has been installed, the amount of water leaking into the pipe is collected and measured, and must be less than the allowable leakage rate described in the *Standard Specifications*.

Low Pressure Air: The section of pipe to be tested is plugged on both ends and compressed air is added until the pipe reaches a certain pressure. The test consists of measuring the time required for the pressure in the test section to drop approximately 1 psi (7 kilopascals). The measured time must be equal to or greater than the required time described in the *Standard Specifications*.

Metal storm sewer pipe will require the same protective coating to resist corrosion as required for culvert pipe. In addition, asphalt coatings may also be required for ungasketed helical seam metal pipes to enable them to pass one of the pressure tests described above. For example, Treatment 1, as described in Section 8-5.3.1 is needed to satisfy the pressure test for an ungasketed helical lock seam pipe. Gasketed helical lock seams and welded remetalized seams are tight enough to pass the pressure test without an asphalt coating, but may still require a coating for corrosion purposes in some areas of the state. Pipe used for storm sewers must be compatible with the structural fill height tables for maximum and minimum amounts of cover shown in Section 8-11.

8-1.5 Sanitary Sewer Pipe

Sanitary sewers consist of pipes and manholes intended to carry either domestic or industrial sanitary wastewater. Any sanitary sewer work on WSDOT projects will usually be replacement or relocation of existing sanitary sewers for a municipal sewer system. Pipe materials will be in accordance with the requirements of the local sewer district. Sanitary wastewater is fairly corrosive regardless of location and pipe materials should be chosen accordingly.

Pressure testing is always required on sanitary sewers to minimize groundwater infiltration or sewer water exfiltration. The testing is performed in accordance with Division 7-17 of the *Standard Specifications*. As with storm sewers, the contractor has the option of conducting an exfiltration, infiltration, or low-pressure air test. The primary difference between the tests for storm sewers versus the tests for sanitary sewers is that the allowable leakage rate for sanitary sewers is less than the allowable leakage rate for storm sewers.

8-2 Pipe Materials

Various types of pipe material are available for each of the applications described in Section 8-1. Each type of material has unique properties for structural design, corrosion/abrasion resistance, and hydraulic characteristics and will be discussed in detail in this section.

8-2.1 Concrete Pipe

8-2.1.1 Concrete Drain Pipe

Concrete drain pipe is non-reinforced and meets the requirements of ASTM C 118. The strength requirements for concrete drain pipe are less than the strength requirements for other types of concrete pipe. Also, concrete drain pipe can be installed without the use of o-ring gaskets or mortar, which tends to permit water movement into and out of the joints.

8-2.1.2 Concrete Underdrain Pipe

Concrete underdrain pipe is perforated, non-reinforced, and meets the requirements of AASHTO M 175. The strength requirements for concrete underdrain pipe are the same as the strength requirements for plain concrete culvert pipe.

8-2.1.3 Concrete Culvert, Storm and Sanitary Sewer Pipe

Concrete culvert, storm, and sanitary sewer pipe can be either plain or reinforced. Plain concrete pipe does not include steel reinforcing and meets the requirements of AASHTO M 86, Class 2 only. Reinforced concrete pipe meets the requirements of AASHTO M 170, Classes I through V. The amount of reinforcement in the pipe increases as the class designation increases. Correspondingly, the structural capacity of the pipe also increases. Due to its lack of strength, Class I reinforced concrete pipe is rarely used and is not listed in the fill height tables of Section 8-11.

The reinforcement placed in concrete pipe can be either circular or elliptical in shape. Elliptically designed reinforcing steel is positioned for tensile loading near the inside of the barrel at the crown and invert, and at the outside of the barrel at the springline. As shown in Figure 8-10.3, a vertical line drawn through the crown and invert is referred to as the minor axis of reinforcement. The minor axis of reinforcement will be clearly marked by the manufacturer, and it is extremely important that the pipe be handled and installed with the axis placed in the vertical position.

Concrete joints utilize rubber o-ring gaskets, allowing the pipe to meet the pressure testing requirements for storm sewer applications. The joints, however, do not have any tensile strength and in some cases can pull apart, as discussed in Section 8-7. For this reason, concrete pipe is not recommended for use on grades over 10 percent without the use of pipe anchors, as discussed in Section 8-8.

Concrete pipe is permitted anywhere in the state, regardless of corrosion zone, pH, or resistivity. It has a smooth interior surface, which gives it a relatively low Manning's roughness coefficient listed in Appendix 4-1. The maximum fill height for concrete pipe is limited to about 30 feet (10 m) or less. However, concrete pipe is structurally superior for carrying wheel loads with very shallow cover. For installations with less than 2 feet (0.6 m) of cover, the only acceptable alternate is concrete pipe. Fill Height Table 8-11.2 lists the appropriate class of pipe that should be specified under these conditions.

Concrete is classified as a rigid pipe, which means that applied loads are resisted primarily by the strength of the pipe material, with some additional support given by the strength of the surrounding bedding and backfill. Additional information regarding the structural behavior of rigid pipes is discussed in Section 8-10.3. It is important during the installation process to insure that the pipe is uniformly supported, in order to prevent point load concentrations from occurring along the barrel or at the joints.

The weight of concrete pipe sometimes makes it difficult to handle during installation and this should be considered on certain sites. Also, in sanitary sewer applications, the build up of hydrogen sulfide could be a concern. The designer should follow the recommendations of the local sewer district or municipality when deciding if concrete pipe is an acceptable alternate at a given location.

An estimate of wall thickness for concrete pipe can be found using a simple rule of thumb. Take the inside diameter in feet and add 1 inch. For example, lets assume we have a 24-inch (2 foot) diameter culvert. Add 1 inch to 2 feet and the estimated wall thickness is 3 inches.

8-2.2 Metal Pipe — General

Metal pipe is available in galvanized steel, aluminized steel, or aluminum alloy. All three types of material can be produced with helical corrugations, annular corrugations or as spiral rib pipe. Galvanized and aluminized steel pipe conform the requirements of AASHTO M 36, while aluminum alloy pipe conforms to the requirements of AASHTO M 196.

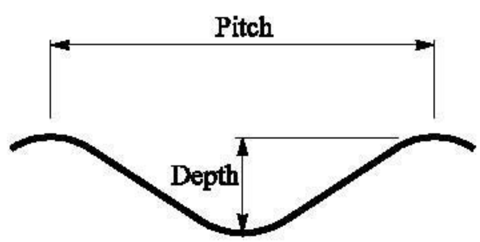
Metal pipe is classified as a flexible pipe, which means that applied loads are resisted primarily by the strength of the bedding and backfill surrounding the pipe, with some additional support given by the pipe material itself. Because of the dependence upon the strength of the bedding and backfill material, it is critical that metal pipe be installed in accordance with the requirements of Standard Plan B-11 and Division 7-08 of the *Standard Specifications* to ensure proper performance.

Metal pipe is available in a wide range of sizes and shapes and, depending on the type of material corrugation configuration, and can be used with fill heights up to 100 feet (30 m) or more. Metal pipe is susceptible to both corrosion and abrasion; methods for limiting these issues are covered in Section 8-5.3 and Section 8-6.

8-2.2.1 Helical Corrugations

Most metal pipe produced today is helically wound, where the corrugations are spiraled along the flow line. The seam for this type of pipe is continuous, and also runs helically along the pipe. The seam can be either an ungasketed lock seam (not pressure testable) or it could be gasketed lock seams (pressure testable seams). If ungasketed lock seam pipe is used in storm sewer applications, it is generally necessary to coat the pipe with Treatment 1 (Section 8-5.3.1) in order for the pipe to pass the pressure testing requirements.

Helically wound corrugations are available in several standard sizes, including 2-2/3 inch pitch by 1/2 inch depth (68 mm pitch by 13 mm depth), 3 inch by 1 inch (75 mm by 25 mm), and 5 inch by 1 inch (125 mm by 25 mm). The corrugation sizes are available in several different gauge thicknesses, depending on the pipe diameter and the height of fill. The larger corrugation sizes tend to be utilized as the pipe diameter exceeds about 60 inch (1500 mm). A typical corrugation section is shown in Figure 8-2.2.1.



Typical Corrugation Section
Figure 8-2.2.1

As a result of the helical manufacturing process, the Manning's roughness coefficient for smaller diameter (less than 24 inch (600 mm)) metal pipe approaches the Manning's roughness coefficient for smooth wall pipe materials such as concrete and thermoplastic pipe. This similarity will generally allow metal pipe to be specified as an alternative to smooth wall pipe without the need to increase the diameter. However, in situations where small changes in the headwater or head loss through a system are critical, or where the pipe diameter is greater than 600 mm (24 in.), the designer should use the Manning's roughness coefficient specified in Appendix 4-1 to determine if a larger diameter metal pipe alternate is required.

8-2.2.2 Annular Corrugations

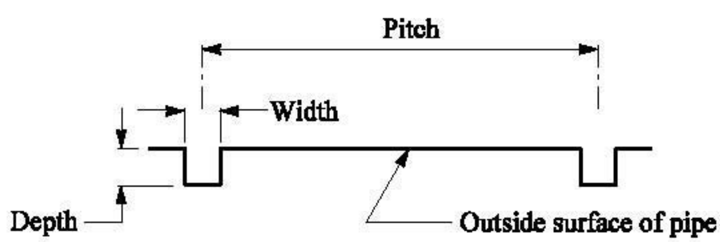
Metal pipe can be produced with annular corrugations, where the corrugations are perpendicular to the flow line of the pipe. The seams for this type of pipe are both circumferential and longitudinal, and are joined by either rivets or resistance spot welding. The Manning's roughness coefficient for all annularly corrugated metal pipe is specified in Appendix 4-1. The fill heights shown in Section 8-11 apply to both helical and annular corrugated metal pipe.

The typical corrugation section shown in Figure 8-2.2.1 is the same for annular corrugations, except that annular corrugations are available only in 2-2/3 inch by 1/2 inch (68 mm by 13 mm) and 3 inch by 1 inch (75 mm by 25 mm) sizes.

8-2.2.3 Spiral Rib

Spiral rib pipe utilizes the same manufacturing process as helically wound pipe, but instead of using a standard corrugation pitch and depth; spiral rib pipe is comprised of rectangular ribs between flat wall areas. A typical spiral rib section is shown in Figure 8-2.2.3. Two profile configurations are available: 3/4 inch width by 3/4 inch depth by 7-1/2 inch pitch (19 mm by 19 mm by 190 mm) or 1 inch by 1 inch by 11 inch (19 mm by 25 mm by 292 mm). The seams for spiral rib pipe are either ungasketed lock seams for non-pressure testable applications or gasketed lock seam for pressure testable applications. If ungasketed lock seam pipe is used in storm sewer applications, it is generally necessary to coat the pipe with protective Treatment 1 (Section 8-5.3.1) in order for the pipe to pass the pressure testing requirements.

The primary advantage of spiral rib pipe is that the rectangular rib configuration provides a hydraulically smooth pipe surface for all diameters, with a Manning's roughness coefficient specified in Appendix 4-1.



Typical Spiral Rib Section
Figure 8-2.2.3

8-2.2.4 Galvanized Steel

Galvanized steel consists of corrugated or spiral rib steel pipe with 1 oz.ft^2 (305 g/m^2) of galvanized coating on each surface of the pipe. Plain galvanized steel pipe is the least durable pipe from a corrosion standpoint and is not permitted when the pH is less than 5 or greater than 8.5. It is also not permitted if the soil resistivity is less than 1,000 ohm-cm. It will, however, meet the required 50-year life expectancy for culvert and storm sewers installed in Corrosion Zone I, as described in Section 8-4. In more corrosive environments, such as Corrosion Zone II or III described in Section 8-4, galvanized steel pipe must be treated with a protective asphalt coating in order for the pipe to attain the required 50-year service life.

8-2.2.5 Aluminized Steel

Aluminized steel consists of corrugated or spiral rib steel pipe with an aluminum protective coating applied both inside and out. The aluminized coating is more resistant to corrosion than galvanized steel pipe and is considered to meet the 50-year life expectancy in both Corrosion Zone I and II without the use of protective asphalt coatings.

Aluminized steel is not permitted when the pH is less than 5 or greater than 8.5. It is also not permitted if the soil resistivity is less than 1,000 ohm-cm.

8-2.2.6 Aluminum Alloy

Aluminum alloy (aluminum) consists of corrugated or spiral rib pipe and has been shown to be more resistant to corrosion than either galvanized or aluminized steel. When aluminum is exposed to water and air, an oxide layer forms on the metal surface, creating a barrier between the corrosive environment and the pipe surface. As long as this barrier is allowed to form, and is not disturbed once it forms, aluminum pipe will function well.

Aluminum is considered to meet the 50-year life expectancy for both Corrosion Zone I and II. It can also be used in Corrosion Zone III, provided that the pH is between 4 and 9, the resistivity is 500 ohm-cm or greater, and the pipe is backfilled with clean, well-draining, granular material. The backfill specified in Standard Plan B-11 will meet this requirement.

Aluminum is not recommended when backfill material has a very high clay content, because the backfill material can prevent oxygen from getting to the pipe surface and consequently, the protective oxide layer will not form. For the same reason, it is generally not recommended that aluminum pipe be coated with the asphalt protective treatments discussed in Section 8-5.3.1

8-2.3 Thermoplastic Pipe — General

Thermoplastic pipe is a term used to describe a number of different types of polyethylene (PE) and polyvinyl chloride (PVC) pipes that are allowed for use in drain, underdrain, culvert, storm sewer, and sanitary sewer applications. Not all types of thermoplastic pipe are allowed for use in all applications. The designer must reference the appropriate section of Division 9-05 of the *Standard Specifications* to determine the allowable thermoplastic pipe for a given application.

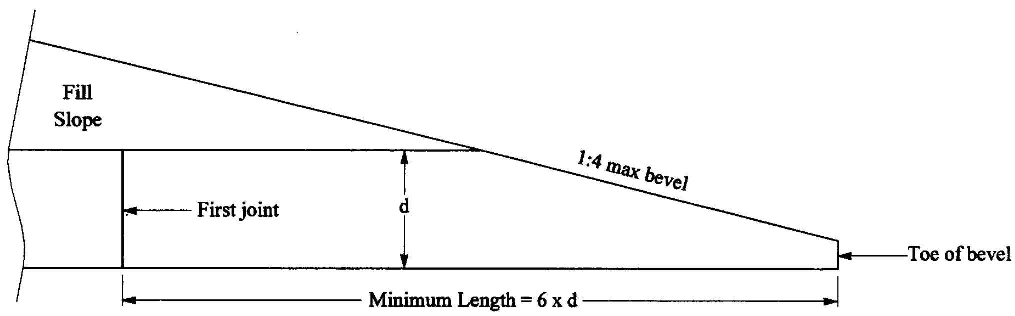
Thermoplastic pipe is classified as a flexible pipe, which means that applied loads are resisted primarily by the strength of the bedding and backfill surrounding the pipe, with some additional support given by the pipe material itself. Because of the dependence upon the strength of the bedding and backfill material, it is critical that thermoplastic pipe be installed in accordance with the requirements of Standard Plan B-11 and Division 7-08 of the *Standard Specifications* to ensure proper performance.

The physical properties of thermoplastic pipe are such that the pipe is very resistant to both pH and resistivity. As a result, thermoplastic pipe is an acceptable alternate in all three corrosion zones statewide and no protective treatment is required. Laboratory testing indicates that the resistance of thermoplastic pipe to abrasive bed loads is equal to or greater than that of other types of pipe material. However, because thermoplastic pipe cannot be structurally reinforced, it is not recommended for severely abrasive conditions as described in Figure 8.6.

The weight of thermoplastic pipe is relatively light when compared to other pipe alternatives. This can simplify handling of the pipe because large equipment may not be necessary during installation. However, the light weight of the pipe can also lead to soil or water floatation problems in the trench, requiring additional effort to secure the line and grade of the pipe.

The allowable fill height and diameter range for thermoplastic pipe is somewhat limited. This may preclude thermoplastic pipe being specified for use in some situations.

Any exposed end of thermoplastic pipe used for culvert or storm sewer applications should be beveled to match the surrounding embankment or ditch slope. The ends should be beveled no flatter than 4:1, as a loss of structural integrity tends to occur after that point. It also becomes difficult to adequately secure the end of the pipe to the ground. The minimum length of a section of beveled pipe shall be at least 6 times the diameter of the pipe, measured from the toe of the bevel to the first joint under the fill slope (see Figure 8-2.3). This distance into the fill slope will provide enough cover over the top of the pipe to counteract typical hydraulic uplift forces that may occur. For thermoplastic pipe 30 inch (900 mm) in diameter and larger, it is recommended that a Standard Plan B-9 headwall be used in conjunction with a beveled end.



Minimum Length for Thermoplastic Pipe Beveled Ends
Figure 8-2.3

8-2.3.1 Corrugated PE Tubing for Drains and Underdrains

Corrugated PE tubing used for drains and underdrains is a single wall, corrugated interior pipe conforming to the requirements of AASHTO M 252. It is available in diameters up to 10 inches (250 mm). This type of pipe is extremely flexible and be manipulated easily on the job site should it become necessary to bypass obstructions during installation. See Section 8-1.1 for treating the exposed end for floatation.

8-2.3.2 PVC Drain and Underdrain Pipe

PVC drain and underdrain pipe is a solid wall, smooth interior pipe conforming to the requirements of AASHTO M 278. It is available in diameters up to 200 mm (8 in.). This type of pipe is typically delivered to the job site in 6 m (20 ft) lengths and has a significant amount of longitudinal beam strength. This characteristic is useful when placing the pipe at a continuous grade but can also make it more difficult to bypass obstructions during installation. See Section 8-1.1 for treating the exposed end for floatation.

8-2.3.3 Corrugated PE Culvert and Storm Sewer Pipe

Corrugated PE used for culverts and storm sewers is a double-wall, smooth interior pipe conforming to the requirements of AASHTO M 294 Type S. This type of pipe can be used under all state highways, subject to the fill height and diameter limits described in Section 8-11 of this manual and Division 7-02.2 of the *Standard Specifications*.

The primary difference between PE used for culvert applications and PE used for storm sewer applications is the type of joint specified. In culvert applications, the joint is not completely watertight and may allow an insignificant amount of infiltration to occur. The culvert joint will prevent soils from migrating out of the pipe zone, and is intended to be similar in performance to the coupling band and gasket required for metal pipe. If a culvert is to be installed in situations where a combination of a high water table and fine-grained soils near the trench are expected, it is recommended that the joint used for storm sewer applications be specified. The storm sewer joint will eliminate the possibility of soil migration out of the pipe zone and will provide an improved connection between sections of pipe.

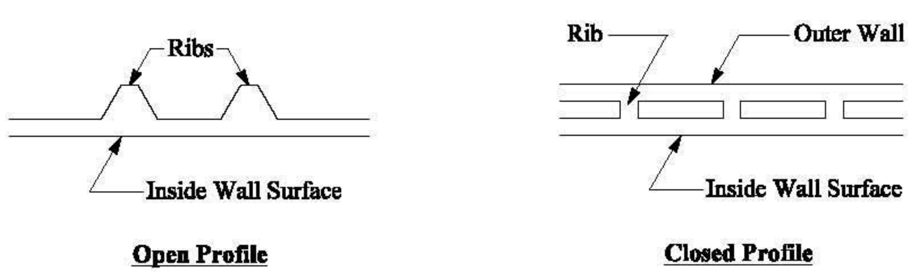
In storm sewer applications, all joints must be capable of passing WSDOT's pressure test requirements. Because of this requirement, it may be possible that the allowable pipe diameter for storm sewer applications may be less than the allowable diameter for culvert applications. The designer should consult WSDOT's Qualified Products List for the current maximum allowable pipe diameter for both applications. Corrugated PE is a petroleum-based product, and it is possible under certain conditions that it will ignite. If maintenance practices such as ditch or field burning is anticipated near the inlet or outlet of a pipe, it is recommended that PE not be allowed as a pipe alternate.

8-2.3.4 Solid Wall PVC Culvert, Storm, and Sanitary Sewer Pipe

Solid wall PVC culvert, storm, and sanitary sewer pipe is a solid wall, smooth interior pipe conforming to the requirements of ASTM D 3034 SDR 35 for pipes up to 15 inches (375 mm) in diameter and ASTM F 679, Type 1 only, for pipe sizes 18 to 27 inch (450 to 625 mm). This type of pipe can be used under all state highways, subject to the fill height and diameter limits described in Section 8-11 of this manual and Divisions 7-02.2 of the *Standard Specifications*. This type of pipe is used primarily in water line and sanitary sewer applications, but may occasionally be used for culverts or storm sewers. The only joint available for this type of PVC pipe is a watertight joint conforming to the requirements of Division 9-05.12(1) of the *Standard Specifications*.

8-2.3.5 Profile Wall PVC Culvert and Storm Sewer Pipe

Profile wall PVC culvert and storm sewer pipe consists of pipe with an essentially smooth waterway wall braced circumferentially or spirally with projections or ribs, as shown in Figure 8-2.3.5. The pipe may have an open profile, where the ribs are exposed, or the pipe may have a closed profile, where the ribs are enclosed in an outer wall. Profile wall PVC culvert and storm sewer pipe must conform to the requirements of AASHTO M 304 or ASTM F794, Series 46. This pipe can be used under all state highways, subject to the fill height and diameter limits described in Section 8-11 of this manual and Divisions 7-02.2 of the *Standard Specifications*. The only joint available for profile wall PVC culvert and storm sewer pipe is a watertight joint conforming to the requirements of Division 9-05.12(2) of the *Standard Specifications*.



Typical Profile Wall PVC Cross Sections
Figure 8-2.3.5

8-2.4 Ductile Iron Pipe

Ductile iron pipe is an extremely strong, durable pipe primarily designed to be used in high-pressure water distribution and sanitary sewer systems. It is acceptable to use ductile iron for culvert and storm sewers, but it is generally not a cost-effective option. Fill heights for ductile iron can be obtained from various manufacturers or by contacting the HQ Hydraulics Branch.

8-3 Pipe Alternates

The plans and special provisions for each project shall be developed to provide the contractor the option of all the alternates that are feasible and in conformance with this manual, the *Standard Specifications*, and WSDOT's *Standard Plans for Road, Bridge, and Municipal Construction*. The appropriate alternatives for a given site should be determined after the designer has investigated a number of issues, including but not limited to the required size and strength of the pipe, corrosion and abrasion potential, fish passage, debris passage, and necessary end treatments.

A methodology for determining the acceptable pipe alternates based on the corrosion potential for a location is presented in Section 8-4. Justification for not providing a pipe alternate, as limited by the allowable fill heights, corrosion zones, soil resistivity, and the limitations of pH for steel and aluminum pipe shall be submitted on a project by project basis in the Hydraulic Report or with the PS&E.

When drain, culvert, or sewer pipe is being constructed for the benefit of cities or counties as part of the reconstruction of their facilities and they request a certain type of pipe, the designer may specify a particular type without alternates; however, the city or county must submit a letter stating their justification. Existing culverts should be extended with the same pipe material and no alternates are required.

Justification for a specific material is required if no alternates are provided. Justification according to cost will not normally be sufficient except in large structures such as box culverts or structural plate pipes. Frequently, structural requirements may have more control over acceptable alternates than will hydraulic requirements.

The hydraulic design for pipe alternates will normally be based on providing equivalent headwater, head loss, or velocity through the pipe. If the design utilized a smooth wall pipe, the use of corrugated pipe may require that the diameter be increased to compensate for the increased surface roughness. The increased pipe diameter for the corrugated pipe alternative shall be listed on the plan sheets.

All pipe used for culverts, storm sewers, and sanitary sewers shall be installed in accordance with Standard Plan B-11. As shown on the plan, the volume and type of bedding material required varies, depending on the type of pipe material used. In order to simplify measurement and payment during construction, all costs associated with furnishing and installing the bedding and backfill material within the pipe zone are included in the unit contract price of the pipe.

8-4 Pipe Corrosion Zones and Pipe Alternate Selection

In addition to designing a pipe system to meet both structural and hydraulic requirements, pipe durability must also be investigated to ensure that the pipe design life will be reached. Pipe durability can be evaluated by determining the corrosion and abrasion potential of a given site and then choosing the appropriate pipe material and protective treatment for that location.

In order to simplify this process, the state of Washington has been divided into three corrosion zones, based upon the general corrosive characteristics of that particular zone. A map delineating the three zones is shown in Figure 8-4. A flow chart and corresponding acceptable pipe alternate list have been developed for each of the corrosion zones and are shown in Figures 8-4.1 to 8-4.3. The flow chart and pipe alternate list summarize the information discussed in Section 8-5 related to corrosion, pH, resistivity, and protective treatments and can be used to easily develop all of the acceptable pipe alternates for a given location.

The flow charts and pipe alternate lists do not account for abrasion, as bed loads moving through pipes can quickly remove asphalt coatings applied for corrosion protection. If abrasion is expected to be significant at a given site, the guidelines discussed in Figure 8-6 should be followed.

The designer should always keep in mind the degree of difficulty that will be encountered in replacing a pipe at a future date. Drain pipes are placed relatively shallow and are easy to replace. Culverts tend to have a deeper amount of cover and also pass under the highway alignment making them more difficult to replace. Storm sewers are generally utilized in congested urban areas with significant pavement cover, high traffic use, and a multitude of other buried utilities in the same vicinity. For these reasons, storm sewers are generally considered to be the most expensive and most difficult to replace. These are generalities that will serve as guidelines to the designer. When special circumstances exist (i.e., extremely high fills or extremely expensive structure excavation) the designer should use good engineering judgment to justify the cost effectiveness of higher standard of protective treatment.

8-4.1 Corrosion Zone I

With the exceptions noted below, Corrosion Zone 1 encompasses most of Eastern Washington and is considered the least corrosive part of the state. Plain galvanized steel, untreated aluminized steel, aluminum alloy, thermoplastic, and concrete pipe may all be used in Corrosion Zone I. See Figures 8.4.1A and B for a complete listing of all acceptable pipe alternates for culvert and storm sewer applications. Treatment 5 is required for all storm sewers if the seams are not pressure testable (ungasketed lock seam).

Parts of Eastern Washington, which are not in Corrosion Zone I are placed into Corrosion Zone II. They include:

Okanogan Valley

Pend Oreille Valley

Disautel — Nespelem Vicinity

8-4.2 Corrosion Zone II

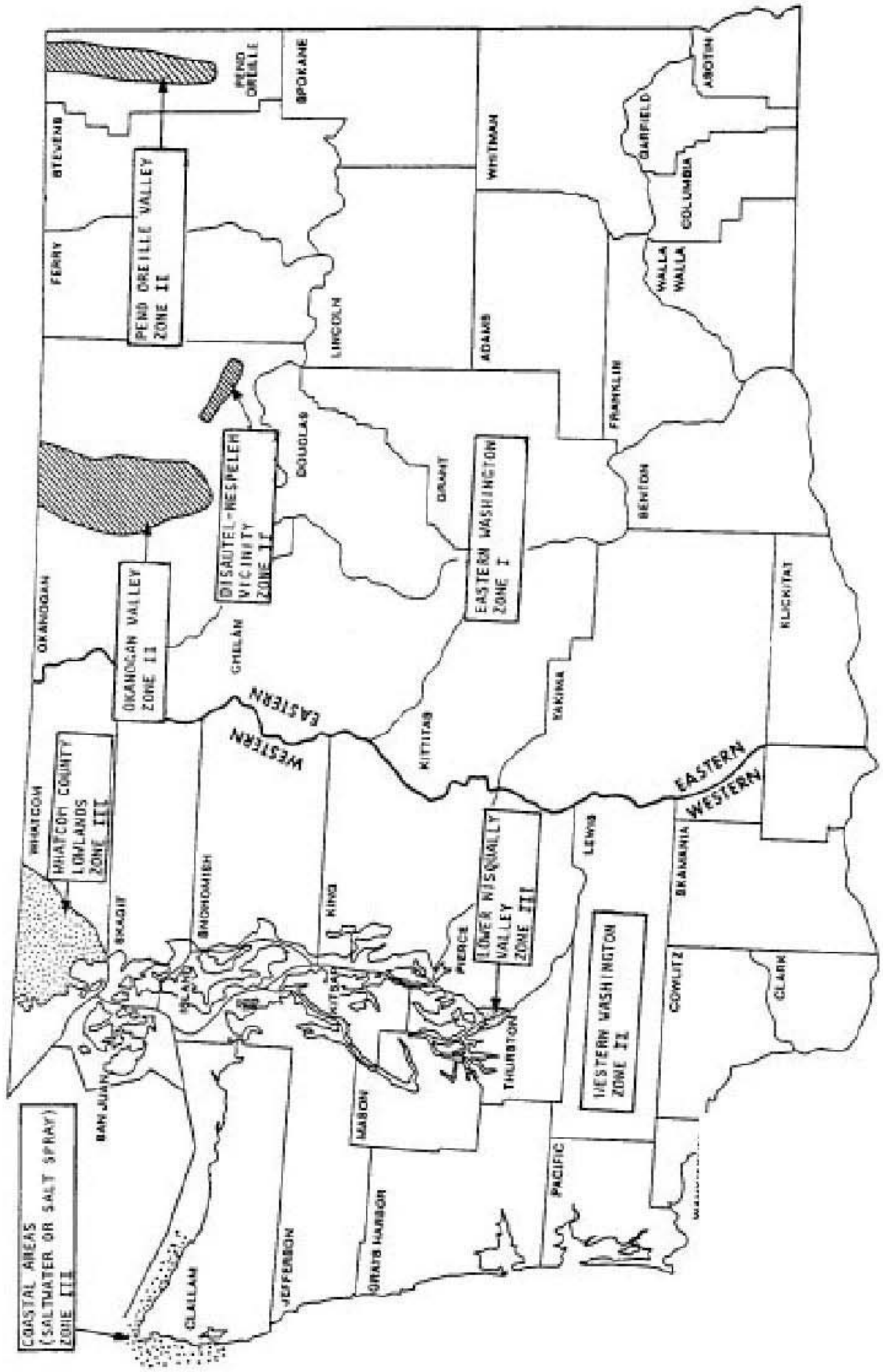
Most of Western Washington, with the exceptions noted below, along with the three areas of Eastern Washington identified above make up Corrosion Zone II. This is an area of moderate corrosion activity. Generally, Treatment 2 is the minimum needed to provide corrosion protection for galvanized steel culverts and storm sewers. Untreated aluminized steel, aluminum alloy, thermoplastic, and concrete pipe may be used in Corrosion Zone II. See Figures 8.4.2A and B for a complete listing of all acceptable pipe alternates for culvert and storm sewer applications.

Parts of western Washington, which are not located in Corrosion Zone II, are placed into Corrosion Zone III. They include:

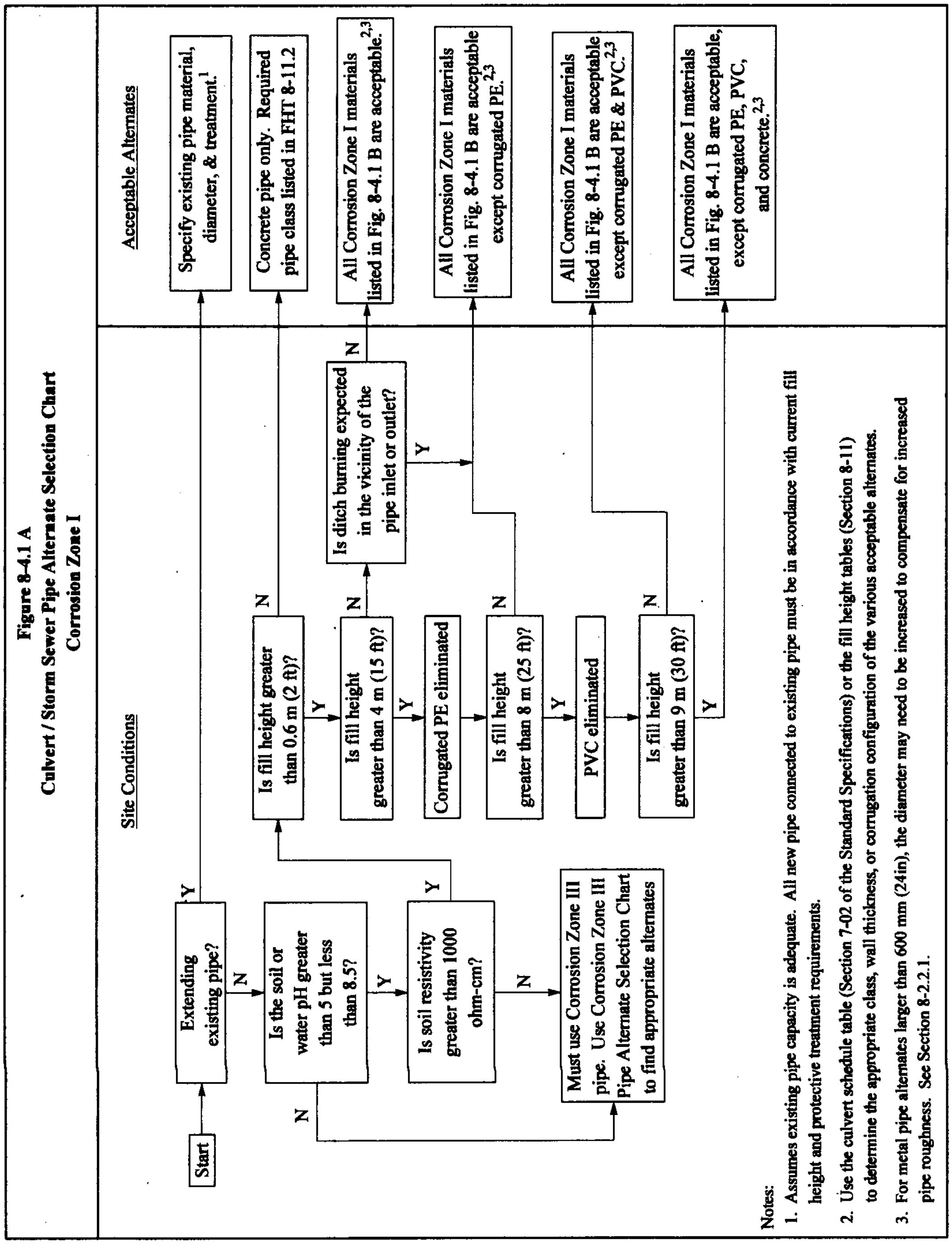
1. Whatcom County Lowlands, described by the following:
    a. SR 542 from its origin in Bellingham to the junction of SR 9;
    b. SR 9 from the junction of SR 542 to the International boundary; and
    c. All other roads and areas lying northerly and westerly of the above described routes.
2. Lower Nisqually Valley.
3. Low-lying roadways in the Puget Sound basin and coastal areas subjected to the influence of saltwater bays, marshes, and tide flats. As a general guideline, this should include areas with elevations less than 20 feet (6 meters) above the average high tide elevation. Along the Pacific coast and the Straits of Juan de Fuca, areas within 300 to 600 feet (100 to 200 meters) of the edge of the average high tide can be influenced by salt spray and should be classified as Corrosion Zone III. However, this influence can vary significantly from location to location, depending on the roadway elevation and the presence of protective bluffs or vegetation. In these situations, the designer is encouraged to evaluate existing pipes in the vicinity of the project to determine the most appropriate corrosion zone designation.

8-4.3 Corrosion Zone III

The severely corrosive areas identified above make up Corrosion Zone III. Concrete and thermoplastic pipe are allowed for use in this zone without protective treatments. Aluminum alloy is permitted only as described in Section 8-2.2.6. See Figures 8.4.3A and B for a complete listing of all acceptable pipe alternates for culvert and storm sewer applications.



Pipe Corrosion Zones
Figure 8-4

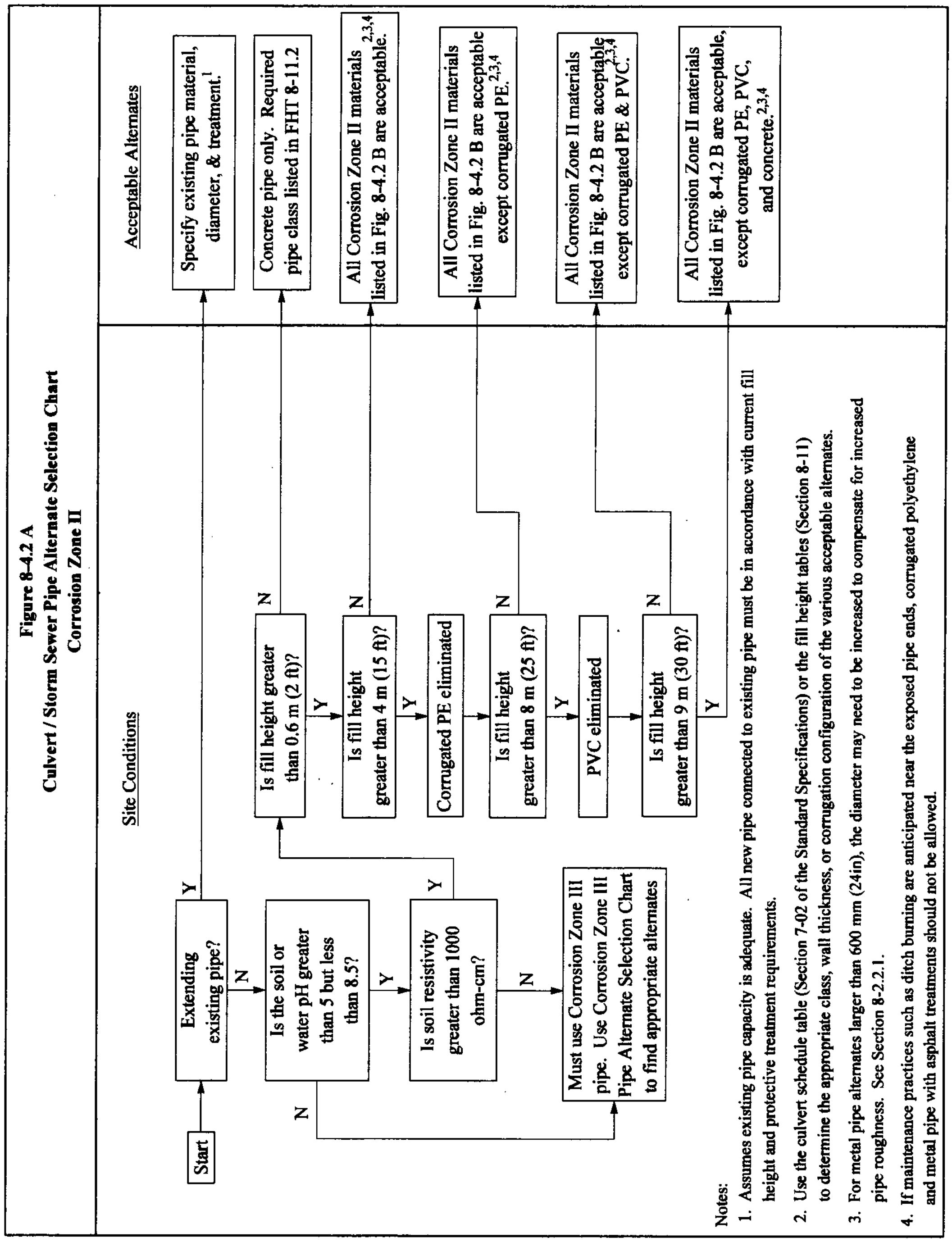


Corrosion Zone I
Culvert/Storm Sewer Pipe Alternate Selection Chart
Figure 8-4.1A

Culverts	Storm Sewers
Schedule Pipe: Schedule____ Culv. Pipe If schedule pipe not selected, then: ***Concrete:*** • Plain Concrete Culvert Pipe • Cl.____Reinf. Concrete Culvert Pipe ***PVC:*** • Solid Wall PVC Culv. Pipe • Profile Wall PVC Culv. Pipe ***Polyethylene:*** • Corrugated Polyethylene Culv. Pipe ***Steel:*** • Plain Galvanized Steel Culv. Pipe • Plain Aluminized Steel Culv. Pipe ***Aluminum:*** • Plain Aluminum Culv. Pipe	***Concrete:*** • Plain Conc. Storm Sewer Pipe • Cl.____Reinf. Conc. Storm Sewer Pipe ***PVC:*** • Solid Wall PVC Storm Sewer Pipe • Profile Wall PVC Storm Sewer Pipe ***Polyethylene:*** • Corrugated Polyethylene Storm Sewer Pipe ***Steel:*** • Tr. 5 Galvanized Steel Storm Sewer Pipe • Plain Galvanized Steel Storm Sewer Pipe with gasketed or welded and remetalized seams • Tr. 5 Aluminized Steel Storm Sewer Pipe • Plain Aluminized Steel Storm Sewer Pipe with gasketed or welded and remetalized seams ***Aluminum:*** • Tr. 5 Aluminum Storm Sewer Pipe • Plain Aluminum Storm Sewer Pipe with gasketed seams ***Steel Spiral Rib:*** • Tr. 5 Galvanized Steel Spiral Rib Storm Sewer Pipe • Plain Galvanized Steel Spiral Rib Storm Sewer Pipe with gasketed or welded and remetalized seams • Tr. 5 Aluminized Steel Spiral Rib Storm Sewer Pipe • Plain Aluminized Steel Spiral Rib Storm Sewer Pipe with gasketed or welded and remetalized seams ***Aluminum Spiral Rib:*** • Tr. 5 Aluminum Spiral Rib Storm Sewer Pipe • Plain Aluminum Spiral Rib Storm Sewer Pipe with gasketed seams

Corrosion Zone I
Acceptable Pipe Alternates and Protective Treatments

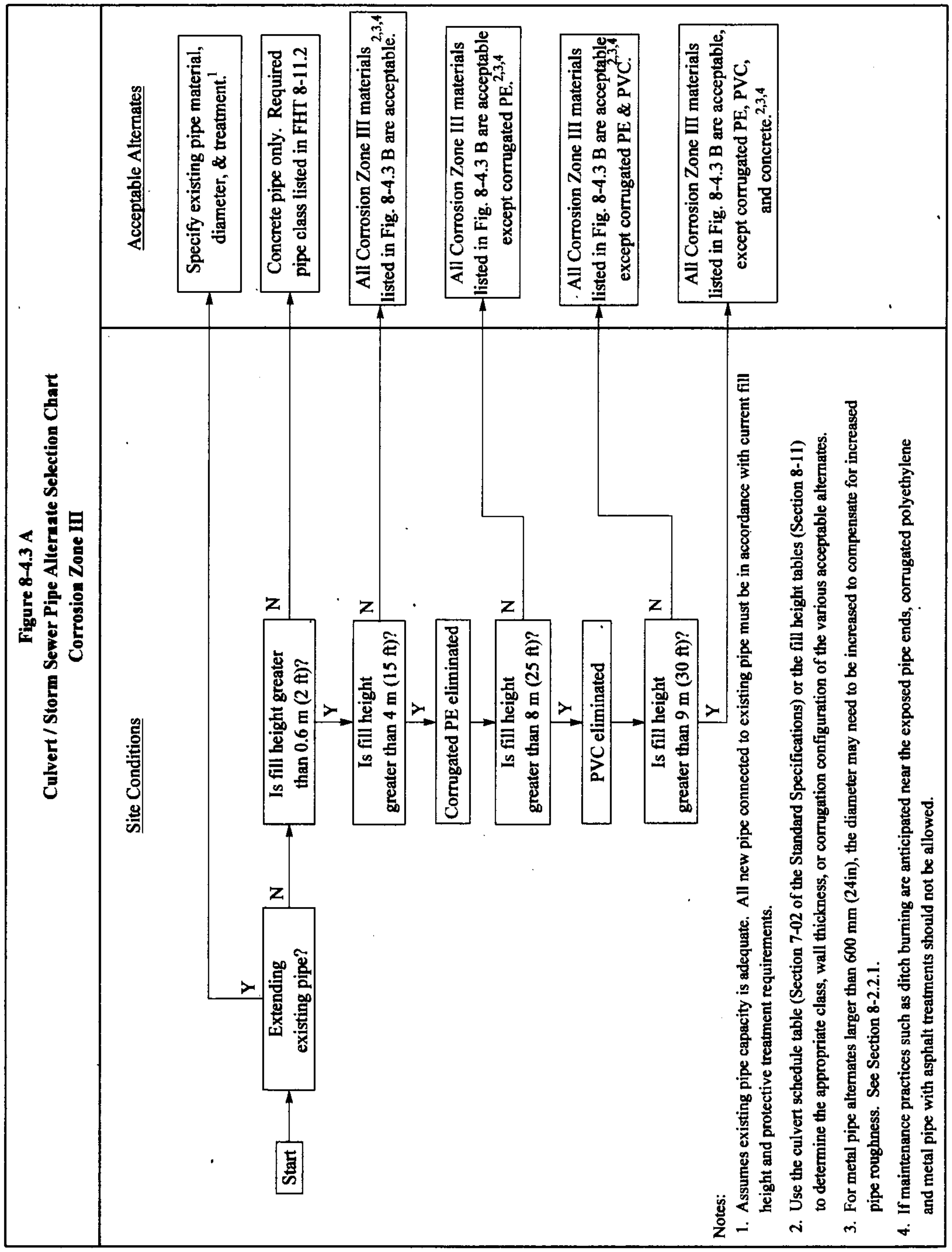
Figure 8-4.1B



Corrosion Zone II
Culvert/Storm Sewer Pipe
Figure 8-4.2A

Culverts	**Storm Sewers**
Schedule Pipe: Schedule____ Culv. Pipe Galvanized Steel alternate shall have Tr. 2 If schedule pipe not selected: ***Concrete:*** • Plain Conc. Culv. Pipe • Cl.____Reinf. Conc. Culv. Pipe ***PVC:*** • Solid Wall PVC Culv. Pipe • Profile Wall PVC Culv. Pipe ***Polyethylene:*** • Corrugated Polyethylene Culv. Pipe ***Steel:*** • Tr. 2 Galvanized Steel Culv. Pipe • Plain Aluminized Steel Culv. Pipe ***Aluminum:*** • Plain Aluminum Culv. Pipe	***Concrete:*** • Plain Conc. Storm Sewer Pipe • Cl.____Reinf. Conc. Storm Sewer Pipe ***PVC:*** • Solid Wall PVC Storm Sewer Pipe • Profile Wall PVC Storm Sewer Pipe ***Polyethylene:*** • Corrugated Polyethylene Storm Sewer Pipe ***Steel:*** • Tr. 5 Galvanized Steel Storm Sewer Pipe • Tr. 2 Galvanized Steel Storm Sewer Pipe with gasketed or welded and remetalized seams • Tr. 5 Aluminized Steel Storm Sewer Pipe • Plain Aluminized Steel Storm Sewer Pipe with gasketed or welded and remetalized seams ***Aluminum:*** • Tr. 5 Aluminum Storm Sewer Pipe • Plain Aluminum Storm Sewer Pipe with gasketed seams ***Steel Spiral Rib:*** • Tr. 5 Galvanized Steel Spiral Rib Storm Sewer Pipe • Tr. 2 Galvanized Steel Spiral Rib Storm Sewer Pipe with gasketed or welded and remetalized seams • Tr. 5 Aluminized Steel Spiral Rib Storm Sewer Pipe • Plain Aluminized Steel Spiral Rib Storm Sewer Pipe with gasketed or welded and remetalized seams ***Aluminum Spiral Rib:*** • Tr. 5 Aluminum Spiral Rib Storm Sewer Pipe • Plain Aluminum Spiral Rib Storm Sewer Pipe with gasketed seams

Corrosion Zone II
Acceptable pipe Alternates and Protective Treatments
Figure 8-4.2B



Corrosion Zone III
Culvert/Storm Sewer Pipe Alternate Selection Chart
Figure 8-4.3A

Culverts	**Storm Sewers**
Schedule Pipe: Schedule____ Culv. Pipe____ In. Diam. Galvanized Steel alternate shall have Tr. 4 Aluminum alternate shall have Tr. 2 If schedule pipe not selected: ***Concrete:*** • Plain Conc. Culv. Pipe • Cl.____Reinf. Conc. Culv. Pipe ***PVC:*** • Solid Wall PVC Culv. Pipe • Profile Wall PVC Culv. Pipe ***Polyethylene:*** • Corrugated Polyethylene Culv. Pipe ***Steel:*** • Tr. 3, Tr. 4, or Tr. 6 Galvanized Steel Culv. Pipe ***Aluminum:*** • Plain Aluminum Culv. Pipe*	***Concrete:*** • Plain Conc. Storm Sewer • Cl.____Reinf. Conc. Storm Sewer Pipe ***PVC:*** • Solid Wall PVC Storm Sewer Pipe • Profile Wall PVC Storm Sewer Pipe ***Polyethylene:*** • Corrugated Polyethylene Storm Sewer Pipe ***Steel:*** • Tr. 6 Galvanized Steel Storm Sewer Pipe • Tr. 4 Galvanized Steel Storm Sewer Pipe with gasketed or welded and remetalized seams ***Aluminum:*** • Tr. 5 Aluminum Storm Sewer Pipe • Plain Aluminum Storm Sewer Pipe with gasketed seams* ***Steel Spiral Rib:*** • Tr. 6 Galvanized Steel Spiral Rib Storm Sewer Pipe • Tr. 4 Galvanized Steel Spiral Rib Storm Sewer Pipe with gasketed or welded and remetalized seams ***Aluminum Spiral Rib:*** • Tr. 5 Aluminum Spiral Rib Storm Sewer Pipe • Plain Aluminum Spiral Rib Storm Sewer Pipe with gasketed seams* * Can be used if the requirements of Section 8-2.2.6 are met

Corrosion Zone III
Acceptable Pipe Alternates and Protective Treatments
Figure 8-4.3B

8-5 Corrosion

Corrosion is the destructive attack on a material by a chemical or electrochemical reaction with the surrounding environment. Corrosion is generally limited to metal pipes, and the parameters that tend to have the most significant influence on the corrosion potential for a site is the soil or water pH and the soil resistivity.

8-5.1 pH

The pH is a measurement of the relative acidity of a given substance. The pH scale ranges from 1 to 14, with 1 being extremely acidic, 7 being neutral, and 14 being extremely basic. The closer a pH value is to 7, the less potential the pipe has for corroding. When the pH is less than 5 or greater than 8.5, the site will be considered unsuitable and only Corrosion Zone III pipes as discussed in Section 8-4.3 are acceptable.

The total number of pH tests required for a project will vary depending on a number of different parameters including: the type of structures to be placed, the corrosion history of the site, and the project length and location. The general criteria listed below serves as minimum guidelines for determining the appropriate number of tests for a project.

1. Size and importance of the drainage structure - A project comprised of large culverts or storm sewers under an interstate or other major arterial warrant testing at each culvert or storm sewer location, while a project comprised of small culverts under a secondary highway may only need a few tests for the entire length of project.
2. Corrosion history of the project location - A site in an area of the state with a high corrosion potential would warrant more tests than a site in an area of the state with a low corrosion potential.
3. Distance of the project - Longer projects tend to pass through several different soil types and geologic conditions, increasing the likelihood of variable pH readings. Tests should be taken at each major change in soil type or topography, or in some cases, at each proposed culvert location. Backfill material that is not native to the site and that will be placed around metal pipe should also be tested.
4. Initial testing results - If initial pH tests indicate that the values are close to or outside of the acceptable range of 5 to 8.5, or if the values vary considerably from location to location, additional testing may be appropriate.

8-5.2 Resistivity

Resistivity is the measure of the ability of soil or water to pass electric current. The lower the resistivity value, the easier it is for the soil or water to pass current, resulting in increased corrosion potential. If the resistivity is less than 1,000 Ohm-cm for a location, then Corrosion Region III pipe materials are the only acceptable alternates. Resistivity test are usually performed in conjunction with pH tests, and the criteria for frequency of pH testing shall apply to resistivity testing as well.

8-5.3 Methods for Controlling Corrosion

8-5.3.1 Protective Treatments

Metal pipe, depending on the material and the geographical location, may require a protective asphalt coating to insure corrosion resistance throughout the pipe design life. As a general guideline, research has shown that asphalt coatings can typically add 15 to 35 years of life to metal pipes. Listed below are three different protective asphalt treatments available for use. The material specifications for the protective treatments are described in Division 9-05.4(3), (4) and (6) of the *Standard Specifications*.

Treatment 1: Coated uniformly inside and out with asphalt. This treatment will protect the soil side of the pipe from corrosion but will only protect the water side of the pipe from corrosion in environments that have little or no bed load moving through the pipe. Most culverts and storm sewers experience some degree of bed load, whether it is native upstream material or roadway sanding debris. The abrasive characteristics of the bed load can remove the asphalt coating relatively quickly, eliminating any corrosion resistance benefit. Consequently, this treatment is rarely specified.

Treatment 2: Coated uniformly inside and out with asphalt and with an asphalt paved invert. This treatment differs from Treatment 1 in that the invert of the pipe is paved with asphalt. Normal water levels within a pipe generally encompass about 40 percent of the circumference of the pipe, and this is where most of the corrosion takes place. The inside coating of the pipe above the normal watermark is not usually attacked by corrosion. Below the normal watermark, the protective coating suffers from wet and dry cycles and is also exposed to abrasion. For these reasons, the bottom 40 percent of the pipe is most critical and, therefore, paved with asphalt.

Treatment 3: No longer available.

Treatment 4: No longer available.

Treatment 5: Coated uniformly inside and out with asphalt and a 100 percent periphery inside spun asphalt lining. This treatment coats the entire inside circumference of the pipe with a thick layer of asphalt, covering the inside corrugations and creating a hydraulically smooth (see Manning's value in Appendix 4-1) interior. The coating also provides invert protection similar to Treatment 2. Treatment 5 can be used on ungasketed lock seam pipe to seal the seam and allow the pipe to pass a pressure test in storm sewer applications.

Treatment 6: No longer available.

The protective treatments, when required, shall be placed on circular pipe as well as pipe arch culverts. Structural plate pipes do not require protective treatment as described in Section 8-1.3.3. Protective treatments are not allowed for culverts placed in fish bearing streams. This may preclude the use of metal culverts in some applications.

The treatments specified in this section are the standard minimum applications, which are adequate for a large majority of installations, however a more stringent treatment may be used at the designers discretion. When unusual abrasive or corrosive conditions are anticipated and it is difficult to determine which treatment would be adequate, it is recommended that either the HQ Materials Laboratory or HQ Hydraulics Branch be consulted.

8-5.3.2 Increased Gauge Thickness

As an alternative to asphalt protective treatments, the thickness of corrugated steel pipes can be increased to compensate for loss of metal due to corrosion. A methodology has been developed by California Transportation Department (Caltrans) to estimate the expected service life of untreated corrugated steel pipes. The method utilizes pH, resistivity, and pipe thickness and is based on data taken from hundreds of culverts throughout California. Copies of the design charts for this method can be obtained from the Regional Hydraulics Section/Contact or from the HQ Hydraulics Branch.

8-6 Abrasion

Abrasion is the wearing away of pipe material by water carrying sands, gravels, and rocks. All types of pipe material are subject to abrasion and can experience structural failure around the pipe invert if not adequately protected. Four abrasion levels have been developed to assist the designer in quantifying the abrasion potential of a site. The abrasion levels are identified in Figure 8-6. The descriptions of abrasion levels are intended to serve as general guidance only, and not all of the criteria listed for a particular abrasion level need to be present to justify placing a site at that level. Included with each abrasion level description are guidelines for providing additional invert protection. The designer is encouraged to use those guidelines in conjunction with the abrasion history of a site to achieve the desired design life of a pipe.

Sampling of the streambed materials is generally not necessary, but visual examination and documentation of the size of the materials in the stream bed and the average stream slopes will give the designer guidance on the expected level of abrasion. Where existing culverts are in place in the same drainage, the condition of the inverts should also be used as guidance. The stream velocity should be based on typical flows, such as a 6-month event, and not a 10- or 50-year event. This is because most of the abrasion will occur during those smaller events.

In streams with significant bed loads, placing culverts on flat grades can encourage bed load deposition within the culvert. This can substantially decrease the hydraulic capacity of a culvert, ultimately leading to plugging or potential roadway overtopping on the upstream side of the culvert. As a standard practice, culvert diameters should be increased two or more standard sizes over the required hydraulic opening in situations where abrasion and bed load concerns have been identified.

Abrasion Level	General Site Characteristics	Recommended Invert Protection
Non Abrasive:	• Little or no bed load • Slopes less than 1% • Velocities less than 1 m/s (3 ft/sec)	Generally, the asphalt protective treatments required for corrosion and specified in Section 8-5.3.1 will provide adequate abrasion protection under these conditions.
Low Abrasive:	• Minor bed loads of sands, silts, and clays • Slopes 1% to 2% • Velocities less than 2 m/s (6 ft/sec)	Generally, the asphalt protective treatments required for corrosion specified in Section 8-5.3.1 will provide adequate abrasion protection under these conditions. An additional gauge thickness of metal pipe may be specified if existing pipes in the same vicinity have demonstrated susceptibility to abrasion.
Moderate Abrasive:	• Moderate bed loads of sands and gravels, with stone sizes up to about 75 mm (3 in.) • Slopes 2% to 4% • Velocities from 2 to 4.5 m/s (6-15 ft/sec)	Metal pipe should be specified with asphalt paved inverts or fiber bonded asphalt. The pipe thickness should be increased one to two standard gages, and the designer should consider concrete-lined metal pipe alternatives. Box culverts should be specified with an increased wall thickness.
Severe Abrasive	• Heavy bed loads of sands, gravel and rocks, with stone sizes up to 300 mm (12 in.) or larger • Slopes steeper than 4% • Velocities greater than 4.5 m/s (15 ft/sec)	Asphalt protective treatments will have extremely short life expectancies, sometimes lasting only a few months to a few years. Metal pipe thickness should be increased at least two standard gages, or the pipe invert should be lined with concrete. Box culverts should be specified with an increased wall thickness or an increased concrete compressive strength. Sacrificial metal plates, channels, or rails may need to be installed in the pipe invert to increase the service life. It is recommended that either the Region Hydraulics Section/Contact or the OSC Hydraulics Branch be contacted for additional guidance when designing this type of invert protection. Thermoplastic pipe exhibits better abrasion characteristics than metal or concrete. However, it generally cannot be reinforced to provide additional invert protection and is not recommended in this condition.

Pipe Abrasion Levels

Figure 8.6

8-7 Pipe Joints

Culverts, storm sewers, and sanitary sewers require the use of gasketed joints to restrict the amount of leakage into or out of the pipe. The type of gasket material varies, depending on the pipe application and the type of pipe material being used. The *Standard Plans and Specifications* should be consulted for specific descriptions of the types of joints, coupling bands, and gaskets for the various types of pipe material.

Corrugated metal pipe joints incorporate the use of a metal coupling band and neoprene gasket that strap on around the outside of the two sections of pipe to be joined. This joint provides a positive connection between the pipe sections and is capable of withstanding significant tensile forces. Because of this, corrugated metal pipes can be placed on slopes up to 20 percent without the use of pipe anchors. For slopes greater than 20 percent, the designer should consider the use of pipe anchors as discussed in Section 8-8. Metal pipe joints work well in culvert applications, but usually do not meet the pressure test requirements for storm sewer applications.

Concrete pipe joints incorporate the use of a rubber o-ring gasket and are held together by friction and the weight of the pipe. Precautions must be taken when concrete pipe is placed on grades greater than 10 percent or in fills where significant settlement is expected, because it is possible for the joints to pull apart. Outlets to concrete pipe must be properly protected from erosion because a small amount of undermining could cause the end section of pipe to disjoin, ultimately leading to failure of the entire pipe system. Concrete joints, because of the o-ring gasket, function well in culvert applications and also consistently pass the pressure testing requirements for storm sewers.

Thermoplastic pipe joints vary from manufacturer to manufacturer, but are generally similar in performance to either the corrugated metal pipe joint or the concrete pipe oint described above. There are currently three types of joints available for thermoplastic pipe. They include:

- Integral bell ends with cleats that positively connect to the spigot end.
- Slip-on bell ends connected with o-ring gaskets on the spigot end.
- Strap-on corrugated coupling bands.

All three types of joints have demonstrated adequate pull-apart resistance, and can generally be used on most highway or embankment slopes without the use of pipe anchors. However, anytime the slope exceeds 20 percent, the designer should consider the use of pipe anchors.

8-8 Pipe Anchors

Pipes anchors are typically needed when pipe is to be placed above ground. This type of installation is rare, but can occur when pipes must be placed on very steep or highly erosive slopes. In these cases, if the necessary pipe diameter is relatively small 10 inch (250 mm), continuous corrugated polyethylene tubing can be used withoutthe need for pipe anchors since there are no joints in the pipe. If larger diameter pipe is required, corrugated metal or thermoplastic pipe should be used in conjunction with galvanized steel pipe anchors. If further information is needed, contact HQ Hydraulics.

8-9 Pipe Rehabilitation

Pipes that have deteriorated over time due to either corrosion or abrasion can significantly affect the structural integrity of the roadway embankment. Once identified, these pipes should be repaired in a timely manner, as failure of the pipe could ultimately result in failure of the roadway. The most common repair method is to remove the existing culvert and replace it with a new one. This method generally requires that all or part of the roadway be closed down for a given amount of time. This may or may not be feasible, depending on the location and importance of the roadway and the size of the pipe structure involved. A number of rehabilitation methods are available which can restore structural integrity to the pipe system while not affecting roadway traffic. Various types of synthetic liners can retrofit the pipe interior, providing structural support. Sliplining is another possibility, where a smaller diameter pipe is inserted into the existing pipe and then backfilled with grout. Tunneling and pipe jacking are also possible, and while typically much more expensive that the other methods, may be the only feasible option for placing pipes under interstates or major arterials. If it is determined that one of these methods may be necessary for a project, the Regional Hydraulics Engineer or the HQ Hydraulics Branch should be consulted for additional information.

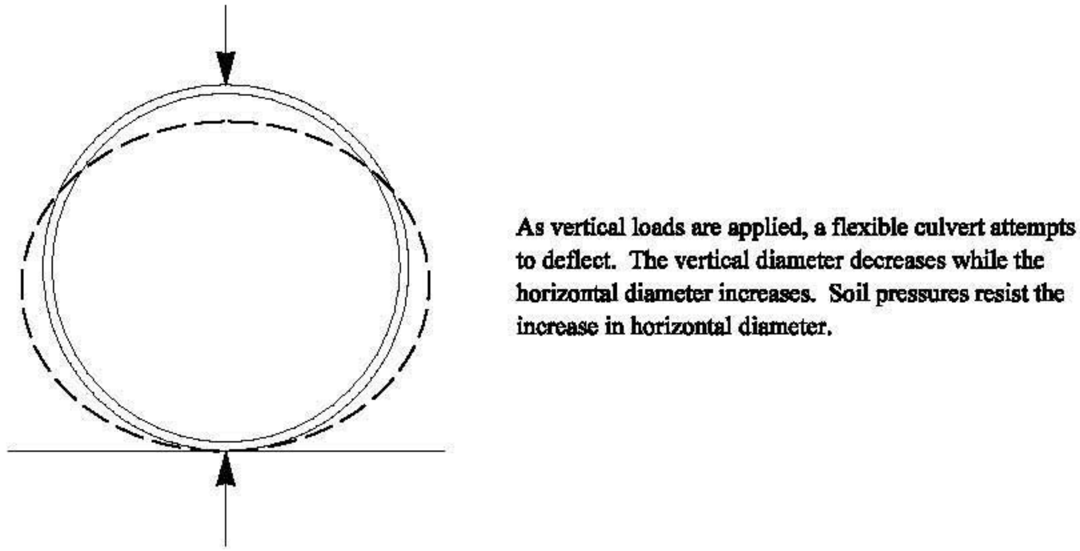
8-10 Pipe Design

8-10.1 Categories of Structural Materials

Based upon material type, pipes can be divided into two broad structural categories: flexible and rigid. Alone, Flexible pipes have little structural bending strength. The material, from which they are made, such as corrugated metal or thermoplastic, can be flexed or distorted significantly without cracking. Consequently, flexible pipes depend on support from the backfill to resist bending. Rigid pipes, however, are stiff and do not deflect appreciably. The material, from which they are made, such as concrete, provides the primary resistance to bending.

8-10.2 Structural Behavior of Flexible Pipes

A flexible pipe is a composite structure made up of the pipe barrel and the surrounding soil. The barrel and the soil are both vital elements to the structural performance of the pipe. Flexible pipe has relatively little bending stiffness or bedding strength on its own. As loads are applied to the pipe, the pipe attempts to deflect. In the case of round pipe, the vertical diameter decreases and the horizontal diameter increases, as shown in Figure 8-10.2. When good backfill material is well compacted around the pipe, the increase in the horizontal diameter of the pipe is resisted by the lateral soil pressure. The result is a relatively uniform radial pressure around the pipe which creates a compressive force in the pipe walls, called thrust. The thrust can be calculated, based on the diameter of the pipe and the load placed on the top of the pipe, and is then used as a parameter in the structural design of the pipe.

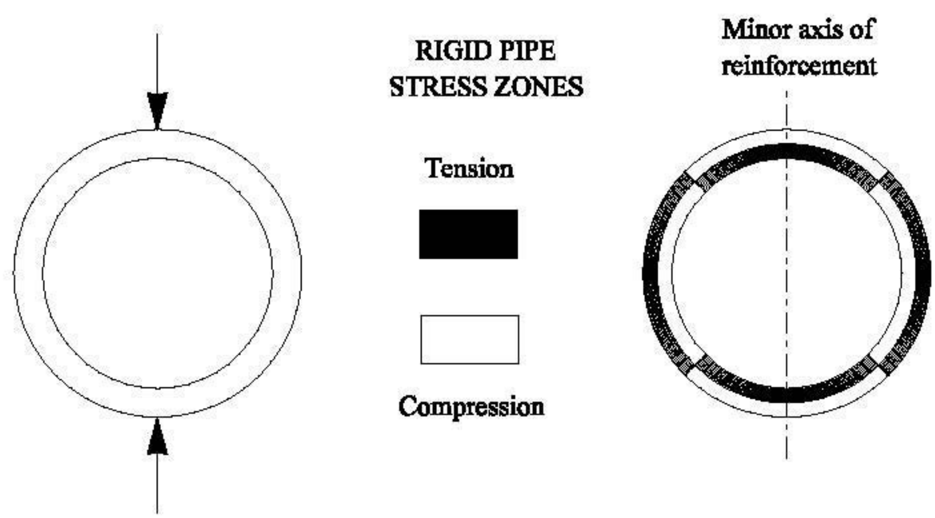


Deflection of Flexible Pipes
Figure 8-10.2

A flexible pipe will be stable as long as adequate soil support is achieved around the pipe. Standard Plan B-11 and Division 7-08 of the *Standard Specifications* describe the backfill material requirements and installation procedures required for placing the various types of pipe materials. Following those guidelines will ensure that a stable soil envelope around the pipe is attained during construction.

8-10.3 Structural Behavior of Rigid Pipes

The load carrying capability of rigid pipes is essentially provided by the structural strength of the pipe itself, with some additional support given by the surrounding bedding and backfill. When vertical loads are applied to a rigid pipe, zones of compression and tension are created as illustrated in Figure 8-10.3. Reinforcing steel can be added to the tension zones to increase the tensile strength of concrete pipe. The minor axis for elliptical reinforcement is discussed in Section 8-2.1.



Zones of Tension and Compression in Rigid Pipes
Figure 8-10.3

Rigid pipe is stiffer than the surrounding soil and it carries a substantial portion of the applied load. Shear stress in the haunch area can be critical for heavily loaded rigid pipe on hard foundations, especially if the haunch support is inadequate. Standard Plan B-11 and Division 7-08 of the *Standard Specifications* describe the backfill material requirements and installation procedures required for placing the various types of pipe materials. The fill height tables for concrete pipe shown in Section 8-11 were developed assuming that those requirements were followed during installation.

8-10.4 Foundation, Bedding, and Backfill

A foundation capable of providing uniform and stable support is important for both flexible and rigid pipes. The foundation must be able to uniformly support the pipe at the proposed grade and elevation without concentrating the load along the pipe. Establishing a suitable foundation requires removal and replacement of any hard spots or soft spots that would result in load concentration along the pipe. Bedding is needed to level out any irregularities in the foundation and to insure adequate compaction of the backfill material. When using flexible pipes, the bedding should be shaped to provide support under the haunches of the pipe. When using rigid pipe, the bedding should be shaped to provide uniform support under the haunches and also shaped to provide clearance for the bell ends on bell and spigot type pipe. The importance of proper backfill for flexible and rigid pipe is discussed in Section 8-10.2 and 8-10.3 respectively. In addition to providing structural support for a pipe, the bedding and backfill must be installed properly to prevent piping from occurring. Piping is a term used to describe the movement of water around and along the outside of a pipe, washing away backfill material that supports the pipe. Piping is primarily a concern in culvert applications, where water at the culvert inlet can saturate the embankment and move into the pipe zone. Piping can be prevented through the use of headwalls, dikes, or plugs. Headwalls are described in Section 3-4-4 and dikes and plugs are discussed in Division 7-02.3(1) of the *Standard Specifications*.

8-10.5 Structural Analysis and Fill Height Tables

The HQ Hydraulics branch, using currently accepted design methodologies, has performed a structural analysis for the various types of pipe material available. The results are shown in the fill height tables of Section 8-11. The fill height tables demonstrate the maximum and minimum amounts of cover that can be placed over a pipe, assuming that the pipe is installed in accordance with WSDOT specifications. All culverts, storm sewers, and sanitary sewers shall be installed within the limitations shown in the fill height tables. The designer shall specify the same wall thickness or class of material for the entire length of a given pipe, and that will be based on the most critical load configuration experienced by any part of the pipe. This will negate the necessity of removing structurally inadequate pipe sections at some point in the future should roadway widening occur. For fill heights in excess of 100 feet (30 m), special designs by the HQ Hydraulics Branch will be required.

8-10.6 Pipe Cover

The amount of cover over the top of a pipe is defined as the distance from the top of the crown of the pipe to the bottom of the pavement. It does not include any asphalt or concrete paving above the top course. The minimum amount of cover for most pipe material is typically 2 feet (0.6 m), but can be less for concrete pipe as described in Section 8-10.7. Unless the contract plans specify a specific pipe material, the designer should design for the schedule pipe fill heights as described in Division 7 of the *Standard Specifications*.

During construction, more restrictive fill heights are required, and are specified in Division 1-07.7 of the *Standard Specifications*. The restrictive fill heights are intended to protect pipe from construction loads that can exceed typical highway design loads.

8-10.7 Shallow Cover Installation

Pipe systems should be designed to provide at least 2 feet (0.6 m) of cover over the top of the pipe. This tends to provide adequate structural distribution of the live load and also allows a significant number of pipe alternatives to be specified on a contract. However, in some cases, it is not possible to lower a pipe profile to obtain the necessary minimum cover. In those cases, only concrete pipe of the class shown in Fill Height Table 8-11.2 should be specified. Included in that table are typical pipe wall thicknesses for a given diameter. The pipe thickness must be taken into consideration in low cover applications. Justification must also be included in the hydraulic report describing why it was not possible to lower the pipe profile to obtain the preferred 2 feet (0.6 m) of cover.

In addition to circular pipe, concrete box culverts and concrete arches are also available for use in shallow cover installations. The designer should consult with either the Regional Hydraulics Section/Contract or the HQ Hydraulics Engineer for additional guidance on the use of these structures in this application.

8-11 Fill Height Tables

Pipe Diameter in.	Maximum Cover in Feet				
	Plain AASHTO M 86	Class II AASHTO M 170	Class III AASHTO M 170	Class IV AASHTO M 170	Class V AASHTO M 170
12	18	10	14	21	26
18	18	11	14	22	28
24	16	11	15	22	28
30		11	15	23	29
36		11	15	23	29
48		12	15	23	29
60		12	16	24	30
72		12	16	24	30
84		12	16	24	30

Minimum Cover: 2 feet

Concrete Pipe
Fill Height Table 8-11.1 (English)

Pipe Diameter mm	Maximum Cover in Meters				
	Plain AASHTO M 86M	Class II AASHTO M 170M	Class III AASHTO M 170M	Class IV AASHTO M 170M	Class V AASHTO M 170M
300	5.5	3.0	4.3	6.5	7.9
450	5.5	3.4	4.3	6.5	8.5
600	5.0	3.4	4.6	6.5	8.5
750		3.4	4.6	7.0	9.0
900		3.4	4.6	7.0	9.0
1200		3.7	4.6	7.0	9.0
1500		3.7	4.9	7.5	9.0
1800		3.7	4.9	7.5	9.0
2100		3.7	4.9	7.5	9.0

Minimum Cover: 0.6 meters

Concrete Pipe
Fill Height Table 8-11.1 (Metric)

Pipe Diameter mm	Pipe Wall Thick. mm	Minimum Cover in Meters			
		Plain AASHTO M 86M	Class III AASHTO M 170M	Class IV AASHTO M 170M	Class V AASHTO M 170M
300	50	0.45	0.45	0.30	0.15
450	63	0.45	0.45	0.30	0.15
600	75	0.45	0.45	0.30	0.15
750	88	0.45	0.45	0.30	0.15
900	100	0.45	0.45	0.30	0.15
1200	125		0.45	0.30	0.15
1500	150		0.45	0.30	0.15
1800	175		0.45	0.30	0.15
2100	200		0.45	0.30	0.15

Concrete Pipe for Shallow Cover Installations
Fill Height Table 8-11.2 (Metric)

Pipe Diameter in.	Pipe Wall Thick. in.	Minimum Cover in Feet			
		Plain AASHTO M 86	Class III AASHTO M 170	Class IV AASHTO M 170	Class V AASHTO M 170
12	2	1.5	1.5	1.0	0.5
18	2.5	1.5	1.5	1.0	0.5
24	3	1.5	1.5	1.0	0.5
30	3.5	1.5	1.5	1.0	0.5
36	4	1.5	1.5	1.0	0.5
48	5		1.5	1.0	0.5
60	6		1.5	1.0	0.5
72	7		1.5	1.0	0.5
84	8		1.5	1.0	0.5

Concrete Pipe for Shallow Cover Installations
Fill Height Table 8-11.2 (English)

Pipe Diameter in.	Maximum Cover in Feet				
	0.064 in. 16 ga	**0.079 in. 14 ga**	**0.109 in. 12 ga**	**0.138 in. 10 ga**	**0.168 in. 8 ga**
12	100	100	100	100	
18	100	100	100	100	
24	98	100	100	100	100
30	78	98	100	100	100
36	65	81	100	100	100
42	56	70	98	100	100
48	49	61	86	100	100
54		54	76	98	100
			68	88	100
				80	98
				73	90
					80
					69

Minimum Cover: 2 feet

Corrugated Steel Pipe
2 2/3in. ´1/2 in. Corrugations
AASHTO M 36
Fill Height Table 8-11.3 (English)

Pipe Diameter mm	Maximum Cover in Meters				
	1.6 mm 16 ga	2.0 mm 14 ga	2.8 mm 12 ga	3.5 mm 10 ga	4.2 mm 8 ga
300	30.5	30.5	30.5	30.5	
450	30.5	30.5	30.5	30.5	
600	30	30.5	30.5	30.5	30.5
750	24	30	30.5	30.5	30.5
900	20	24.5	30.5	30.5	30.5
1050	17	21.5	30	30.5	30.5
1200	15	18.5	26	30.5	30.5
1350		16.5	23	30	30.5
1500			21	27	30.5
1650				24.5	30
1800				22.5	27.5
1950					24.5
2100					21

Minimum Cover: 0.6 meters

Corrugated Steel Pipe
68 mm ´ 13 mm Corrugations
AASHTO M 36M
Fill Height Table 8-11.3 (Metric)

Pipe Diameter in.	Maximum Cover in Feet				
	0.064 in. 16 ga	**0.079 in. 14 ga**	**0.109 in. 12 ga**	**0.138 in. 10 ga**	**0.168 in. 8 ga**
36	75	94	100	100	100
42	64	80	100	100	100
48	56	70	99	100	100
54	50	62	88	100	100
60	45	56	79	100	100
66	41	51	72	92	100
72	37	47	66	84	100
78	34	43	60	78	95
84	32	40	56	72	89
90	30	37	52	67	83
96		35	49	63	77
102		33	46	59	73
108			44	56	69
114			41	53	65
120			39	50	62

Minimum Cover: 2 feet

Corrugated Steel Pipe
3 in. ´ 1 in. Corrugations
AASHTO M 36
Fill Height Table 8-11.4 (English)

Pipe Diameter mm	Maximum Cover in Meters				
	1.6 mm 16 ga	**2.0 mm 14 ga**	**2.8 mm 12 ga**	**3.5 mm 10 ga**	**4.3 mm 8 ga**
900	23	28.5	30.5	30.5	30.5
1050	19.5	24.5	30.5	30.5	30.5
1200	17	21.5	30	30.5	30.5
1350	15	19	27	30.5	30.5
1500	13.5	17	24	30.5	30.5
1650	12.5	15.5	22	28	30.5
1800	11.5	14.5	20	25.5	30.5
1950	10.5	13	18.5	24	29
2100	10	12	17	22	27
2250	9	11.5	16	20.5	25.5
2400		10.5	15	19	23.5
2550		10	14	18	22.5
2700			13.5	17	21
2850			12.5	16	20
3000			12	15	19

Minimum Cover: 0.6 meters

Corrugated Steel Pipe
75 mm ´ 25 mm Corrugations
AASHTO M 36M
Fill Height Table 8-11.4 (Metric)

Pipe Diameter in.	Maximum Cover in Feet				
	0.064 in. 16 ga	**0.079 in. 14 ga**	**0.109 in. 12 ga**	**0.138 in. 10 ga**	**0.168 in. 8 ga**
30	80	100	100	100	100
36	67	83	100	100	100
42	57	71	100	100	100
48	50	62	88	100	100
54	44	55	78	100	100
60	40	50	70	90	100
66	36	45	64	82	100
72	33	41	58	75	92
78	31	38	54	69	85
84	28	35	50	64	79
90	26	33	47	60	73
96		31	44	56	69

Minimum Cover: 2 feet

Corrugated Steel Pipe
5 in. ´ 1 in. Corrugations
AASHTO M 36
Fill Height Table 8-11.5 (English)

Pipe Diameter mm	Maximum Cover in Meters				
	1.6 mm 16 ga	**2.0 mm 14 ga**	**2.8 mm 12 ga**	**3.5 mm 10 ga**	**4.3 mm 8 ga**
750	24.5	30.5	30.5	30.5	30.5
900	20.5	25.5	30.5	30.5	30.5
1050	17.5	21.5	30.5	30.5	30.5
1200	15	19	27	30.5	30.5
1350	13.5	17	24	30.5	30.5
1500	12	15	21.5	27.5	30.5
1650	11	13.5	19.5	25	30.5
1800	10	12.5	17.5	23	28
1950	9.5	11.5	16.5	21	26
2100	8.5	10.5	15	19.5	24
2250	8	10	14.5	18.5	22.5
2400		9.5	13.5	17	21

Minimum Cover: 0.6 meters

Corrugated Steel Pipe
125 mm X 25 mm Corrugations
AASHTO M 36M
Fill Height Table 8-11.5 (Metric)

Pipe Diameter in.	Minimum Cover ft.	Maximum Cover in Feet						
		0.111 in. 12 ga	0.140 in. 10 ga	0.170 in. 8 ga	0.188 in. 7 ga	0.218 in. 5 ga	0.249 in. 3 ga	0.280 in. 1 ga
60	2	42	63	83	92	100	100	100
72	2	35	53	69	79	94	100	100
84	2	30	45	59	67	81	95	100
96	2	27	40	52	59	71	84	92
108	2	23	35	46	53	64	75	81
120	2	21	31	42	47	57	67	74
132	2	19	29	37	42	52	61	66
144	2	18	26	37	40	47	56	61
156	2	16	24	31	36	43	52	56
168	2	15	22	30	33	41	48	53
180	2	14	20	28	31	38	44	49
192	2		19	26	30	35	42	46
204	3		18	24	28	33	40	43
216	3			23	26	31	37	41
228	3				25	30	35	39
240	3				23	29	33	37

Corrugated Steel Structural Plate Circular Pipe
6 in. ´ 2 in. Corrugations
Fill Height Table 8-11.6 (English)

Pipe Diameter mm	Minimum Cover m	Maximum Cover in Meters						
		2.8 mm 12 ga	3.5 mm 10 ga	4.5 mm 8 ga	4.8 mm 7 ga	5.5 mm 5 ga	6.5 mm 3 ga	7.0 mm 1 ga
1500	0.6	13	19	25.5	28	30.5	30.5	30.5
1800	0.6	10.5	16	21	24	28.5	30.5	30.5
2100	0.6	9	13.5	18	20.5	24.5	29	30.5
2400	0.6	8	12	16	18	21.5	22.5	28
2700	0.6	7	10.5	14	16	19.5	23	24.5
3000	0.6	6.5	9.5	13	14.5	17.8	20.5	22.5
3300	0.6	6	9	11.5	13	16	18.5	20
3600	0.6	5.5	8	11.5	12	14.5	17	18.5
3900	0.6	5	7	9.5	11	13	16	17
4200	0.6	4.5	6.5	9	10	12.5	14.5	16
4500	0.6	4.3	6	8.5	9.5	11.5	13.5	15
4800	0.6		6	8	9	10.5	13	14
5100	0.9		5.5	7	8.5	10	12	13
5400	0.9			7	8	9.5	11.5	12.5
5700	0.9				7.5	9	10.5	12
6000	0.9				7	9	10	11.5

Corrugated Steel Structural Plate Circular Pipe
152 mm ´ 51 mm Corrugations
Fill Height Table 8-11.6 (Metric)

Span´ Rise in. ´ in.	Min. Corner Radius in.	Thickness		Minimum Cover Feet	Maximum Cover in Feet for Soil Bearing Capacity of:	
		in.	Gage		2 tons/ft^2	3 tons/ft^2
17 ´ 13	3	0.064	16 ga	2	12	18
21 ´ 15	3	0.064	16 ga	2	10	14
24 ´ 18	3	0.064	16 ga	2	7	13
28 ´ 20	3	0.064	16 ga	2	5	11
35 ´ 24	3	0.064	16 ga	2.5	NS	7
42 ´ 29	3.5	0.064	16 ga	2.5	NS	7
49 ´ 33	4	0.079	14 ga	2.5	NS	6
57 ´ 38	5	0.109	12 ga	2.5	NS	8
64 ´ 43	6	0.109	12 ga	2.5	NS	9
71 ´ 47	7	0.138	10 ga	2	NS	10
77 ´ 52	8	0.168	8 ga	2	5	10
83 ´ 57	9	0.168	8 ga	2	5	10

NS = Not Suitable

Corrugated Steel Pipe Arch
2 in. ´ in. Corrugations
AASHTO M 36
Fill Height Table 8-11.7 (English)

Span´ Rise mm x mm	Min. Corner Radius mm	Thickness		Min. Cover	Maximum Cover in Meters for Soil Bearing Capacity of:	
		mm	Gage	m	191 kPa	290 kPa
430 x 330	75	1.6	16 ga	0.6	3.7	5.5
530 x 380	75	1.6	16 ga	0.6	3	4.3
610 x 460	75	1.6	16 ga	0.6	2.1	4.0
710 x 510	75	1.6	16 ga	0.6	1.5	3.4
885 x 610	75	1.6	16 ga	0.8	NS	2.1
1060 x 740	88	1.6	16 ga	0.8	NS	2.1
1240 x 840	100	2.0	14 ga	0.8	NS	1.8
1440 x 970	125	2.8	12 ga	0.8	NS	2.4
1620 x 1100	150	2.8	12 ga	0.8	NS	2.7
1800 x 1200	175	3.5	10 ga	0.6	NS	3
1950 x 1320	200	4.3	8 ga	0.6	1.5	3
2100 x 1450	225	4.3	8 ga	0.6	1.5	3

NS = Not Suitable

Corrugated Steel Pipe Arch
68 mm ´ 13 mm Corrugations
AASHTO M 36M
Fill Height Table 8-11.7 (Metric)

Span´ Rise in. ´ in.	Corner Radius in.	Thickness		Min. Cover Feet	Maximum Cover in Feet for Soil Bearing Capacity of:	
		in.	Gage		2 tons/ft^2	3 tons/ft^2
40 ´ 31	5	0.079	14 ga	2.5	8	12
46 ´ 36	6	0.079	14 ga	2	8	13
53 ´ 41	7	0.079	14 ga	2	8	13
60 ´ 46	8	0.079	14 ga	2	8	13
66 ´ 51	9	0.079	14 ga	2	9	13
73 ´ 55	12	0.079	14 ga	2	11	16
81 ´ 59	14	0.079	14 ga	2	11	17
87 ´ 63	14	0.079	14 ga	2	10	16
95 ´ 67	16	0.079	14 ga	2	11	17
103 ´ 71	16	0.109	12 ga	2	10	15
112 ´ 75	18	0.109	12 ga	2	10	16
117 ´ 79	18	0.109	12 ga	2	10	15
128´ 83	18	0.138	10 ga	2	9	14
137 ´ 87	18	0.138	10 ga	2	8	13
142 ´ 91	18	0.168	10 ga	2	7	12

Corrugated Steel Pipe Arch
3 in. ´ 1 in. Corrugations
AASHTO M36
Fill Height Table 8-11.8 (English)

Span´ Rise mm ´ mm	Corner Radius mm	Thickness		Min. Cover mm	Maximum Cover in Feet for Soil Bearing Capacity of:	
		mm	Gage		190 kPa	290 kPa
1010 ´ 790	125	2.0	14 ga	0.8	2.4	3.7
1160 ´ 920	150	2.0	14 ga	0.6	2.4	4
1340 ´ 1050	175	2.0	14 ga	0.6	2.4	4
1520 ´ 1170	200	2.0	14 ga	0.6	2.4	4
1670 ´ 1300	225	2.0	14 ga	0.6	2.7	4
1850 ´ 1400	300	2.0	14 ga	0.6	3.4	4.9
2050 ´ 1500	350	2.0	14 ga	0.6	3.4	5.2
2200 ´ 1620	350	2.0	14 ga	0.6	3	4.9
2400 ´ 1720	400	2.0	14 ga	0.6	3.4	5.2
2600 ´ 1820	400	2.8	12 ga	0.6	3	4.5
2840 ´ 1920	450	2.8	12 ga	0.6	3	4.9
2970 ´ 2020	450	2.8	12 ga	0.6	3	4.5
3240 ´ 2120	450	3.5	10 ga	0.6	2.7	4.3
3470 ´ 2220	450	3.5	10 ga	0.6	2.4	4
3600 ´ 2320	450	4.3	8 ga	0.6	2.1	3.7

Corrugated Steel Pipe Arch
75 mm ´ 25 mm Corrugations
AASHTO M-36M
Fill Height Table 8-11.8 (Metric)

Span ´ Rise ft.-in. ´ ft.-in.	Corner Radius in.	Thickness		2 TSF Soil Bearing Capacity		3 TSF Soil Bearing Capacity	
		in.	Gage	Min. Cover ft.	Max. Cover ft.	Min. Cover ft.	Max. Cover ft.
6 – 1 ´ 4 - 7	18	0.111	12 ga	2	16	2	24
7 – 0 ´ 5 - 1	18	0.111	12 ga	2	14	2	21
7 – 11 ´ 5 - 7	18	0.111	12 ga	2	13	2	19
8 – 10 ´ 6 - 1	18	0.111	12 ga	2	11	2	17
9 – 9 ´ 6 - 7	18	0.111	12 ga	2	10	2	15
10 – 11 ´ 7 - 1	18	0.111	12 ga	2	9	2	14
11 – 10 ´ 7 - 7	18	0.111	12 ga	2	7	2	13
12 – 10 ´ 8 - 4	18	0.111	12 ga	2.5	6	2	12
13 – 3 ´ 9 - 4	31	0.111	12 ga	2	13	2	17*
14 – 2 ´ 9 - 10	31	0.111	12 ga	2	12	2	16*
15 – 4 ´ 10 - 4	31	0.140	10 ga	2	11	2	15*
16 – 3 ´ 10 - 10	31	0.140	10 ga	2	11	2	14*
17 – 2 ´ 11 - 4	31	0.140	10 ga	2.5	10	2.5	13*
18 – 1 ´ 11 - 10	31	0.168	8 ga	2.5	10	2.5	12*
19 – 3 ´ 12 - 4	31	0.168	8 ga	2.5	9	2.5	13
19 – 11 ´ 12 - 10	31	0.188	6 ga	2.5	9	2.5	13
20 – 7 ´ 13 - 2	31	0.188	6 ga	3	7	3	13

* Fill limited by the seam strength of the bolts. TSF: tons per square foot

Additional sizes are available. Contact the OSC Hydraulics Branch for more information.

Corrugated Steel Structural Plate Pipe Arch
6 in. ´ 2 in. Corrugations
Fill Height Table 8-11.9 (English)

Span´ Rise mm ´ mm	Corner Radius mm	Thickness		190 kPa Soil Bearing Capacity		290 kPa Soil Bearing Capacity	
		mm	Gage	Min. Cover m	Max. Cover m	Min. Cover m	Max. Cover m
1850 ´ 1400	457	2.8	12 ga	0.6	5	0.6	7
2130 ´ 1550	457	2.8	12 ga	0.6	4.3	0.6	6.5
2410 ´ 1700	457	2.8	12 ga	0.6	4	0.6	6
2690 ´ 1850	457	2.8	12 ga	0.6	3.4	0.6	5
2970 ´ 2010	457	2.8	12 ga	0.6	3	0.6	4.5
3330 ´ 2160	457	2.8	12 ga	0.6	2.7	0.6	4.3
3610 ´ 2310	457	2.8	12 ga	0.6	2.1	0.6	4
3910 ´ 2540	457	2.8	12 ga	0.8	1.8	0.6	3.7
4040 ´ 2840	787	2.8	12 ga	0.6	4	0.6	5
4320 ´ 3000	787	2.8	12 ga	0.6	3.7	0.6	5
4670 ´ 3150	787	3.5	10 ga	0.6	3.4	0.6	4.5
4950 ´ 3300	787	3.5	10 ga	0.6	3.4	0.6	4.3
5230 ´ 3450	787	3.5	10 ga	0.8	3	0.8	4
5510 ´ 3610	787	4.5	8 ga	0.8	3	0.8	3.7
5870 ´ 3760	787	4.5	8 ga	0.8	2.7	0.8	4
6070 ´ 3910	787	4.8	6 ga	0.8	2.7	0.8	4
6270 ´ 4010	787	4.8	6 ga	0.9	2.1	0.9	4

* Fill limited by the seam strength of the bolts.

Additional sizes are available. Contact the OSC Hydraulics Branch for more information.

Corrugated Steel Structural Plate Pipe Arch
152 mm ´ 51 mm Corrugations
Fill Height Table 8-11.9 (Metric)

Pipe Diameter in.	Maximum Cover in Feet				
	0.060 in. (16 ga)	0.075 in. (14 ga)	0.105 in. (12 ga)	0.135 in (10 ga)	0.164 in. (8 ga)
12	100	100			
18	75	94	100		
24	56	71	99		
30		56	79		
36		47	66	85	
42			56	73	
48			49	63	78
54			43	56	69
60				50	62
66					56
72					45

Minimum Cover: 2 Feet

Aluminum Pipe
2 in. ´ in. Corrugations
AASHTO M 196
Fill Height Table 8-11.10 (English)

Pipe Diameter mm	Maximum Cover in Meters				
	1.5 mm (16 ga)	1.9 mm (14 ga)	2.7 mm (12 ga)	3.4 mm (10 ga)	4.2 mm (8 ga)
300	30.5	30.5			
450	23	28.5	30.5		
600	17	21.5	30		
750		56	24		
900		14.5	20	26	
1050			17	22	
1200			15	19	24
1350			13	17	21
1500				15	19
1650					17
1800					13.5

Minimum Cover: 0.6 meters

Aluminum Pipe
68 mm ´ 13 mm Corrugations
AASHTO M 196M
Fill Height Table 8-11.10 (Metric)

Pipe Diameter in.	Maximum Cover in Feet				
	0.060 in. (16 ga)	0.075 in. (14 ga)	0.105 in. (12 ga)	0.135 in. (10 ga)	0.164 in. (8 ga)
36	43	65	76	98	
42	36	46	65	84	
48	32	40	57	73	90
54	28	35	50	65	80
60		32	45	58	72
66		28	41	53	65
72		26	37	48	59
78		24	34	44	55
84			31	41	51
90			29	38	47
96			27	36	44
102				33	41
108				31	39
114					37
120					35

Minimum Cover: 2 Feet

Aluminum Pipe
3 in. ´ 1 in. Corrugations
AASHTO M 196
Fill Height Table 8-11.11 (English)

Pipe Diameter mm	Maximum Cover in Meters				
	1.5 mm (16 ga)	**1.9 mm (14 ga)**	**2.7 mm (12 ga)**	**3.4 mm (10 ga)**	**4.2 mm (8 ga)**
900	13	20	23	30	
1050	11	14	20	25.5	
1200	9.5	12	17.5	22	27.5
1350	8.5	10.5	15	20	24.5
1500		9.5	13.5	17.5	22
1650		8.5	12.5	16	20
1800		8.0	11.5	14.5	18
1950		7.5	10.5	13.5	17
2100			9.5	12.5	15.5
2250			9.0	11.5	14.5
2400			8.0	11	13.5
2550				10	12.5
2700				9.5	12
2850					11.5
3000					10.5

Minimum Cover: 0.6 meters

Aluminum Pipe
75 mm ´ 25 mm Corrugations
Fill Height Table 8-11.11 (metric)

Pipe Dia. in.	Maximum Cover in Feet						
	0.100 in.	**0.125 in.**	**0.150 in.**	**0.175 in.**	**0.200 in.**	**0.225 in.**	**0.250 in.**
60	31	45	60	70	81	92	100
72	25	37	50	58	67	77	86
84	22	32	42	50	58	66	73
96	19	28	37	44	50	57	64
108	17	25	33	39	45	51	57
120	15	22	30	35	40	46	51
132	14	20	27	32	37	42	47
144	12	18	25	29	33	38	43
156		17	23	27	31	35	39
168			31	25	29	33	36
180				23	27	30	34

Minimum Cover: 2 feet

Aluminum Structural Plate
9 in. ´ 2 in. Corrugations With Galvanized Steel Bolts
Fill Height Table 8-11.12 (English)

Pipe Dia. mm.	Maximum Cover in Meters						
	2.5 mm	**3.2 mm**	**3.8 mm**	**4.4 mm**	**5.1 mm**	**5.7 mm**	**6.4 mm**
1500	9.5	13.5	18.5	21.5	24.5	28	30.5
1800	7.5	11.5	15	17.5	20.5	23.5	26
2100	6.5	10	13	15	17.5	20	22.5
2400	6	8.5	11.5	13.5	15	17.5	19.5
2700	5	7.5	10	12	13.5	15.5	17.5
3000	4.5	6.5	9	10.5	12	14	15.5
3300	4.3	6	8	10	11.5	13	14.5
3600	3.7	5.5	7.5	9	10	11.5	13
3900		5	7	8	9.5	10.5	12
4200			6.5	7.5	9	10	11
4500				7	8	9	10.5

Minimum Cover: 0.6 meters

Aluminum Structural Plate
230 mm ´ 64 mm Corrugations With Galvanized Steel Bolts
Fill Height Table 8-11.12 (Metric)

Span ´ Rise in. ´ in.	Corner Radius in.	Thickness		Min. Cover Feet	Maximum Cover in Feet for Soil Bearing Capacity of:	
		in.	Gage		2 tons/ft2	3 tons/ft2
17 ´ 13	3	0.060	16 ga	2	12	18
21 ´ 15	3	0.060	16 ga	2	10	14
24 ´ 18	3	0.060	16 ga	2	7	13
28 ´ 20	3	0.075	14 ga	2	5	11
35 ´ 24	3	0.075	14 ga	2.5	NS	7
42 ´ 29	3.5	0.105	12 ga	2.5	NS	7
49 ´ 33	4	0.105	12 ga	2.5	NS	6
57 ´ 38	5	0.135	10 ga	2.5	NS	8
64 ´ 43	6	0.135	10 ga	2.5	NS	9
71 ´ 47	7	0.164	8 ga	2	NS	10

NS = Not Suitable

Aluminum Pipe Arch
2 2/3´ 1/2 Corrugations
Fill Height Table 8-11.13 (English)

Span ´ Rise mm ´ mm	Corner Radius mm	Thickness		Min. Cover m	Maximum Cover in Meters for Soil Bearing Capacity of:	
		mm	Gage		190 kPa	290 kPa
430 ´ 330	75	1.5	16 ga	0.6	3.7	5.5
530 ´ 380	75	1.5	16 ga	0.6	3	4.3
610 ´ 460	75	1.5	16 ga	0.6	2.1	4
710 ´ 510	75	1.9	14 ga	0.6	1.5	3.4
885 ´ 610	75	1.9	14 ga	0.8	NS	2.1
1060 ´ 740	89	2.7	12 ga	0.8	NS	2.1
1240 ´ 840	102	2.7	12 ga	0.8	NS	1.8
1440 ´ 970	127	3.4	10 ga	0.8	NS	2.4
1620 ´ 1100	152	3.4	10 ga	0.8	NS	2.7
1800 ´ 1200	178	4.2	8 ga	0.6	NS	3.0

NS = Not Suitable

Aluminum Pipe Arch
68 mm ´ 13 mm Corrugations
AASHTO M 196M
Fill Height Table 8-11.13 (Metric)

Span ´ Rise in. ´ in.	Corner Radius in.	Thickness		Min. Cover Feet	Maximum Cover in Feet for Soil Bearing Capacity of:	
		in.	Gage		2 tons/ft2	3 tons/ft2
40 ´ 31	5	0.075	14 ga	2.5	8	12
46 ´ 36	6	0.075	14 ga	2	8	13
53 ´ 41	7	0.075	14 ga	2	8	13
60 ´ 46	8	0.075	14 ga	2	8	13
66 ´ 51	9	0.060	14 ga	2	9	13
73 ´ 55	12	0.075	14 ga	2	11	16
81 ´ 59	14	0.105	12 ga	2	11	17
87 ´ 63	14	0.105	12 ga	2	10	16
95 ´ 67	16	0.105	12 ga	2	11	17
103 ´ 71	16	0.135	10 ga	2	10	15
112 ´ 75	18	0.164	8 ga	2	10	16

Aluminum Pipe Arch
3 ´ 1 Corrugations
AASHTO M 196
Fill Height Table 8-11.14 (English)

Span ´ Rise mm ´ mm	Corner Radius mm	Thickness		Min. Cover m	Maximum Cover in Feet for Soil Bearing Capacity of:	
		mm	Gage		190 kPa	290 kPa
1010 ´ 790	127	1.9	14 ga	0.8	2.4	3.7
1160 ´ 920	152	1.9	14 ga	0.6	2.4	4
1340 ´ 1050	178	1.9	14 ga	0.6	2.4	4
1520 ´ 1170	203	1.9	14 ga	0.6	2.4	4
1670 ´ 1300	229	1.9	14 ga	0.6	2.7	4
1850 ´ 1400	305	1.9	14 ga	0.6	3.4	5
2050 ´ 1500	356	1.7	12 ga	0.6	3.4	5
2200 ´ 1620	356	2.7	12 ga	0.6	3	5
2400 ´ 1720	406	2.7	12 ga	0.6	3.4	5
2600 ´ 1820	406	3.4	10 ga	0.6	3	4.5
2840 ´ 1920	457	4.2	8 ga	0.6	3	5

Aluminum Pipe Arch
75 mm ´ 25 mm Corrugations
AASHTO M 196M
Fill Height Table 8-11.14 (Metric)

Span ´ Rise ft - in ´ ft - in		Corner Radius in.	Minimum Gage Thickness in.	Min. Cover ft.	Maximum Cover (1) in Feet For Soil Bearing Capacity of:	
					2 tons/ft2	3 tons/ft2
a	5 – 11 x 5 – 5	31.8	0.100	2	24*	24*
b	6 – 11 x 5 –9	31.8	0.100	2	22*	22*
c	7 – 3 x 5 – 11	31.8	0.100	2	20*	20*
d	7 – 9 x 6 – 0	31.8	0.100	2	28*	18*
e	8 – 5 x 6 – 3	31.8	0.100	2	17*	17*
f	9 – 3 x 6 – 5	31.8	0.100	2	15*	15*
g	10 – 3 x 6 – 9	31.8	0.100	2	14*	14*
h	10 – 9 x 6 – 10	31.8	0.100	2	13*	13*
i	11 – 5 x 7 –1	31.8	0.100	2	12*	12*
j	12 – 7 x 7 – 5	31.8	0.125	2	14	16*
k	12 – 11 x 7 – 6	31.8	0.150	2	13	14*
l	13 – 1 x 8 – 2	31.8	0.150	2	13	18*
m	13 – 11 x 8 – 5	31.8	0.150	2	12	17*
n	14 – 8 x 9 – 8	31.8	0.175	2	12	18
o	15 – 4 x 10 – 0	31.8	0.175	2	11	17
p	16 – 1 x 10 – 4	31.8	0.200	2	10	16
q	16 – 9 x 10 – 8	31.8	0.200	2.17	10	15
r	17 – 3 x 11 – 0	31.8	0.225	2.25	10	15
s	18 – 0 x 11 – 4	31.8	0.255	2.25	9	14
t	18 – 8 x 11 – 8	31.8	0.250	2.33	9	14

*Fill limited by the seam strength of the bolts.

(1) Additional sizes and varying cover heights are available, depending on gage thickness and reinforcement spacing. Contact the OSC Hydraulics Branch for more information.

Aluminum Structural Plate Pipe Arch
9 in. ´ 2 2/3 in. Corrugations, ¼ in. Steel Bolts, 4 Bolts/Corrugation
Fill Height Table 8-11.15 (English)

Span ´ Rise mm ´ mm		Corner Radius mm	Minimum Gage Thickness mm	Min. Cover m	Maximum Cover (1) in Feet for Soil Bearing Capacity of:	
					190 kPa	290 kPa
a	1800´ 1650	808	2.5	0.6	7*	7*
b	2100´ 1750	808	2.5	0.6	6.5*	6.5*
c	2210´ 1800	808	2.5	0.6	6*	6*
d	2360´ 1830	808	2.5	0.6	5.5*	5.5*
e	2570´ 1910	808	2.5	0.6	5*	5*
f	2820´ 1960	808	2.5	0.6	4.5*	4.5*
g	3120´ 2060	808	2.5	0.6	4.3*	4.3*
h	3280´ 2080	808	2.5	0.6	4*	4*
i	3480´ 2160	808	2.5	0.6	3.7*	3.7*
j	3840´ 2260	808	3.2	0.6	4.3	5*
k	3940´ 2290	808	3.8	0.6	4	4.3*
l	3990´ 2490	808	3.8	0.6	4	5.5*
m	4240´ 2570	808	3.8	0.6	3.7	5*
n	4470´ 2950	808	4.4	0.6	3.7	5.5
o	4670´ 3050	808	4.4	0.6	3.4	5
p	4900´ 3150	808	5.1	0.6	3	5
q	5110´ 3250	808	5.1	0.67	3	4.5
r	5260´ 3350	808	5.7	0.69	3	4.5
s	5490´ 3450	808	6.4	0.69	2.7	4.3
t	5690´ 3560	808	6.4	0.71	2.7	4.3

*Fill limited by the seam strength of the bolts.

(1) Additional sizes and varying cover heights are available, depending on gage thickness and reinforcement spacing. Contact the OSC Hydraulics Branch for more information.

Aluminum Structural Plate Arch
230 mm ´ 64 mm Corrugations, 19 mm Steel Bolts, 4 Bolts/Corrugation
Fill Height Table 8-11.15 (Metric)

Diameter in.	Maximum Cover in Feet		
	0.064 in. 16 ga	**0.079 in. 14 ga**	**0.109 in. 12 ga**
18	50	72	
24	50	72	100
30	41	58	97
36	34	48	81
42	29	41	69
48	26	36	61
54	21	32	54
60	19	29	49

Minimum Cover: 2 feet

Steel and Aluminized Steel Spiral Rib Pipe
3/4 ´ 1 ´ 11 1/2 in. or ¾ ´ ¾ ´ 7 1/2 in. Corrugations
AASHTO M 36
Fill Height Table 8-11.16 (English)

Diameter mm	Maximum Cover in Meters		
	1.6 mm 16 ga	**2.0 mm 14 ga**	**2.8 mm 12 ga**
450	15	22	
600	15	22	30.5
750	12.5	17.5	29.5
900	10.5	14.5	24.5
1050	9	12.5	21
1200	8	11	18.5
1350	7	10	16.5
1500	6	9	15

Minimum Cover: 0.6 meters

Steel and Aluminized Steel Spiral Rib Pipe
19 ´ 25 ´ 292 mm or 19 ´ 19 ´ 191 mm Corrugations
AASHTO M 36M
Fill Height Table 8-11.16 (Metric)

Diameter in.	Maximum Cover in Feet			
	0.060 in. 16 ga	0.075 in. 14 ga	0.105 in. 12 ga	0.135 10 ga
12	35	50		
18	34	49		
24	25	36	63	82
30	19	28	50	65
36	15	24	41	54
42		19	35	46
48		17	30	40
54		14	27	35
60		12	24	30

Minimum Cover: 2 feet

Aluminum Alloy Spiral Rib Pipe
´ 1 ´ 11 in. or ´ ´ 7 in. Corrugations
AASHTO M 196
Fill Height Table 8-11.17 (English)

Diameter mm	Maximum Cover in Meters			
	1.5 mm 16 ga	1.9 mm 14 ga	2.7 mm 12 ga	3.4 mm 10 ga
300	11	15		
450	10.5	14.5		
600	7.5	11	19	25
750	6	8.5	15	20
900	4.5	7.5	12.5	16.5
1050		6	10.5	14
1200		5	9	12
1350		4.3	8	10.5
1500		3.7	7.5	9

Minimum Cover: 0.6 meters

Aluminum Alloy Spiral Rib Pipe
19 ´ 25 ´ 292 mm or 19 ´ 19 ´ 190 mm Corrugations
AASHTO M 196M
Fill Height Table 8-11.17 (Metric)

Solid Wall PVC	Profile Wall PVC	Corrugated Polyethylene
ASTM D 3034 SDR 35 3 in. to 15 in. dia. ASTM F 679 Type 1 18 in. to 27 in. dia.	AASHTO M 304 or ASTM F 794 Series 46 4 in. to 48 in. dia.	AASHTO M 294 Type S 12 in. to 36 in. dia.
25 feet All diameters	25 feet All diameters	15 feet All diameters

Minimum Cover: 2 feet

Thermoplastic Pipe (English)
Fill Height Table 8-11.18

Solid Wall PVC	Profile Wall PVC	Corrugated Polyethylene
ASTM D 3034 SDR 35 75 mm to 375 mm dia. ASTM F 679 Type 1 450 mm to 675 mm dia.	AASHTO M 304 or ASTM F 794 Series 46 100 mm to 1200 mm dia.	AASHTO M 294 Type S 300 mm to 900 mm dia.
8 meters All diameters	8 meters All diameters	4 meters All diameters

Minimum Cover: 0.6 meters

Thermoplastic Pipe (Metric)
Fill Height Table 8-11.18

9-1 General

Each region is responsible for the design of the water and sewage disposal systems for rest areas, maintenance buildings, and other capital improvements. Upon request from the region, Olympia Service Center (OSC) Hydraulics Branch can complete the design in its entirety if given the appropriate information. In any case, OSC Hydraulics Branch will be available for design guidance. The hydraulic report for this type of project is a Type A report.

Because the designs of an adequate potable water supply and an adequate sewage disposal system are required for determination of the full right of way needs, they must be reviewed and approved by the OSC Hydraulics Branch in advance of the finalization of the ultimate right of way plans.

Adequate planning and design of water supply and sewage disposal systems require consultation and coordination with OSC Field Operations Support Service Center - Roadside Maintenance Branch and the appropriate federal, state, and local agencies during the earliest stages of planning. Consult with the Central Operations Division of the Department of Ecology (DOE) regarding the need of a water right permit, and with the local health agency or the Department of Health (DOH), for a source site inspection when developing the water supply. When the proposed facilities for rest areas include an on-site sewage disposal system, consult with the local health authority, the DOH or DOE, depending on the design flow: see Section 9-4.3.

9-2 Submittal

The regions shall submit site plans and the data outlined in Sections 9-2.1 and 9-2.2 in a bound report to the OSC Hydraulics Branch prior to finalization of the right of way plans.

Water supply and wastewater disposal facilities must obtain the approval of the appropriate regulatory agencies: Department of Ecology; Department of Health, Environmental Health Programs; and/or the local health department.

The Rules and Regulations of the Department of Health for public water supplies has specific design requirements and recommended guidelines: see WAC 246-290.

9-2.1 Water Supply System — Data Requirements

1. Source, location, and type of electrical power. (Single or three phase and voltages available.)
2. Geologist report to include:
    a. Depth of water-bearing strata determined by test drilling.
    b. Requirement of gravel-pack.
    c. Size of well screen required.
    d. Pump test for quantity. DOH requires a minimum 4-hour test.
    e. Health Department test for quality.
3. Engineering calculations for proposed design.
4. Results of consultation with DOE for water rights permit, and with DOH concerning source location.

5. Recommendations concerning restroom capacity.
6. Irrigation requirements in terms of highest anticipated flow rate and approximate area to be irrigated.

9-2.2 Sewage Disposal System — Data Requirements

1. Site plan of sewage disposal system.
2. Engineering report to include:
    a. Recommended type of disposal system, such as septic tank and drainfield, leaching bed, sewage lagoon, septic tank effluent pressure line, etc.
    b. Soil types in area of drainfield, lagoon, etc.
    c. Depth of groundwater.
    d. Results of soil investigation conducted in the method(s) prescribed by the State and County Health Departments. Detail the data to include pit site locations, soil logs and the date completed.
    e. Design criteria.
3. Engineering calculations for proposed design.

9-3 Water Supply

Most highway rest areas are located in rural areas and therefore require an on-site well to provide the water supply. In cases where the rest area is in the service area of a private or public water system, the designer must investigate the costs of water service from the water district in lieu of a well. Consideration should be given to the on-going maintenance costs involved in operating a well.

9-3.1 Test Well

If an on-site well is required to provide the water supply, a test well must be drilled to ensure an adequate supply of potable water. The test well may be drilled under a well drilling contract or a fully operated equipment agreement as described in RCW 47.28.030. The right of way for the test well may be purchased in accordance with RCW 47.12.011. The agreement should include permission to enter the land in order to drill a well and conduct the necessary tests. After an adequate water supply has been obtained, the right of way may be acquired.

The final determination of each pump capacity should be made after the capacity of the well has been determined, so that the pump capacity will not greatly exceed the well capacity.

Wells drilled for highway rest areas shall have a minimum diameter of 8 inches. Details for well design can be found in *Minimum Standards for Construction and Maintenance of Wells - Chapter 173-160 WAC,* prepared by DOE.

9-3.2 Water Demands at Rest Areas

The design criteria used by the Washington State Department of Transportation (WSDOT) for major rest areas are specific to the rest area. Through traffic counts, OSC Transportation Data Office (TDO) can calculate the size of the rest area facility. TDO will determine the percent of the average daily traffic entering the rest area on an average day and a peak day, the percent of vehicles entering during the peak hour, the average number of occupants per vehicle, the percentage of occupants using the rest room and the annual growth rate for the area. From this information, TDO calculates the number of parking stalls and restroom units required. Their analysis will include current and design year criteria. Only in the absence of any historical demand information should the values given in Figure 9-3.2 be used.

Average Daily Traffic (ADT) projected for 20 years hence.

Rest area on interstate; percent of ADT entering on an average day	12%
Rest area on interstate; percent of ADT entering on a peak day	24%
Rest area near parks, resorts, or towns; percent of ADT entering on an average day	5%
Percent of people stopping usingrest-rooms	77%
Average number of people per vehicle	2.2
Average water use per person, m^3 (gallons)	0.013 (3.5) Conventional toilet 0.013 (3.5) Conventional urinal 0.002 (0.5) Compressed air toilet
Peak hour factor; as a percent of ADT entering the rest area	12%

Rest Area Design Criteria
Figure 9-3.2

Most existing rest areas have historical water use records to aid the designer in developing the proper usage rates. The designer should be aware of the hourly distribution and water demand throughout the design day or design weekend. Summer holiday weekends nearly always control the size of the water systems and shall be used for the design of water storage tanks and pump capacities, septic tanks, and drain field sizes for rest areas.

Some systems may be sensitive to the monthly variations in rest area usage. Septic tank and drainfields and sewage lagoons may require the designer to consider these monthly variations in demand along with climatic variations.

Another consideration that should be included in the design of a rest area is the potential for a recreational vehicle (RV) dump site. A breakdown of the number and type of RV's using the rest area should be requested from the TDO. The type of RV is of importance because the volume of wastewater generated from an RV varies from 0.095 m^3 to 0.378 m^3 (25 gallons to 100 gallons) dependent on whether the RV is a camper, trailer, or motor home. If an RV type classification is not provided, the designer should assume for sewage disposal purposes that each RV will dispose of 0.3 m^3 (80 gallons) of wastewater. When estimating the amount of water required by an RV at the dump site, 0.15 m^3 (40 gallons) per RV should be used.

9-3.2.1 User Analysis

Once the design criteria is known, the supply of water necessary to meet rest area demands can be determined from the number of users.

Dependent on the detail of user information available, there are several ways in which the required water supply can be determined. From general to detailed, the water supply can be determined in terms of the average daily traffic, the number of vehicles using the rest area, or the number of people using the rest rooms. In most cases, detailed information is not available and the analysis is based on the average daily traffic. For discussion purposes, the three approaches are discussed below. Using the data provided by TDO or Figure 9-3.2, the equation for calculating the daily water usage from average daily traffic counts is as follows:

Volume in m^3 used on an average day = AxBxCxDxE

where A = Average Daily Traffic (ADT)

B = Percent vehicles entering rest area

C = Number of persons per vehicle

D = Percent people using rest rooms

E = Water use per person, m^3 (gallons)

Using the numerical values in Figure 9-3.2, as an example, the equation becomes:

Volume in m^3 used on an average day = (ADT) (0.12) (2.2) (0.77) (0.013)

Volume in m^3 used on an average day = (ADT) (0.003)

(Gallons used on an average day = ADT 0.712)

In general, the peak hour usage should be checked to ensure that this demand can be met. This can be done by including a peak hour factor multiplier, (F) This factor is a percent of the ADT entering the rest area. The formula is then revised to read:

Volume in m^3 used during peak hour = AxBxCxDxExF

Once again, using the numerical values in Figure 9-3.2, as an example, the equation becomes:

Volume in m^3 used during peak hour = (ADT) (0.12) (2.2) (0.77) (0.013) (0.12)

Volume in m^3 used during peak hour = (ADT) (3.2×10^{-4})

(Gallons used during peak hour = ADTx0.085)

The second method in determining the supply of water required is based on the number of vehicles using the rest area. This method assigns the number of gallons required per vehicle. For example, on average, one person using the rest room uses 0.013 m^3 (3.5 gallons) of water. This amount may be less if low flow toilets or sinks are used. If 80 percent of the people in the vehicles use the rest room and there is an average of 3 people per vehicle, one could say that the occupants of one vehicle use $3.0 \times 0.80 \times 0.013 = 0.031$ m^3 (8.4 gallons). If the number of vehicles using the rest area is known, the total amount of water needed can be determined.

Similarly, for RV dumps, the water supply required would be equivalent to the number of RV's using the dump times the average amount of water required per RV, 0.15 m^3 (40 gallons). For example, if 67 RV's used the dump site in a day, the daily total water supply required would be 10 m^3 (2,680 gallons).

The third method is based on the number of times the facilities are used. At times, the number of people using the rest rooms may be known. The total water supply required to meet rest room demands is the number of users per day times 0.013 m^3 (3.5 gallons) per user. For example, a rest area that has 9,300 users/day will need 123.2 m^3 (32,550 gallons) of water to meet rest room demands. When using this method in estimating the required water supply, it is important to include all possible uses of the facilities, such as rest rooms, RV dumps, and irrigation.

Similar to the first method, the daily water supply determined from the second and third method can be expressed in terms of peak and future use. In general the design should be based on a 20-year design. The volumes needed in the design year can be calculated by applying an annual growth rate to the amount estimated under existing conditions. Peak hour usage can also be figured by applying a peak hour factor. Regardless of which time frame is being used, the designer should assume that the amount of water consumed by rest room usage equals the sewage generated. In the case of sewage from RV dumps, the designer should assume the sewage generated is 0.3 m^3 (80 gallons) per RV or twice the water consumed.

9-3.3 Reservoirs

The supply of water from on-site wells alone may not be sufficient to meet peak demands. Storage facilities such as reservoirs and pressure tanks are useful in satisfying these demands. An elevated storage reservoir in lieu of a pressure tank should always be considered in the design of the water supply, especially where the reservoir could be conveniently located on a nearby hill. Elevated storage reservoirs are preferred because these reservoirs can operate on a gravity system, reducing maintenance and o peration costs.

An underground reservoir should be considered when a low producing well is being utilized. This will allow for storage which will meet the maximum demand without exceeding the capacity of the well.

When an underground reservoir is used, two pumps can be specified, one for supplying the irrigation water and the other for supplying the domestic water. If a separate source of water supply is used for irrigation, it must be plainly identified as such and not be interconnected with the potable supply system.

All pressure tanks shall be designed using the *Sizing Guidelines for Public Water Supplies* , September 1983, from the Department of Health. All hydropneumatic tanks larger than 0.14 m^3 (37.4 gallons) in volume must be of ASME approved construction and have the ASME identifying plate and data sheets for proper registration.

9-4 Sewage Disposal

The first consideration in the selection of a rest area site should be the suitability of the area to provide an adequate means for sewage disposal. Since the cost for performing the required tests for a drainfield are much less than those required for a well, it seems only reasonable that the sewage disposal system should be given first consideration.

A bound report should be submitted to the Regional or OSC Hydraulics Branch for review and approval. This report should include the information described in Section 9-2.2 along with any other information required for the type of sewage disposal system chosen. The report shall be written in collaboration with the appropriate local or state health agencies.

9-4.1 Municipal Sewer Systems

Whenever a rest area is located near a large populated area served by a municipal sewer system, the designer should give serious consideration to connecting the rest area directly to the sewer by a gravity line. Metering the sewage is best accomplished by metering the incoming water supply and assuming this to be equal to the sewage flows.

All sanitary sewers shall be designed according to the criteria furnished by the local sewer district. In the absence of such criteria or when building on highway right of way, sanitary sewers shall be designed and constructed to give mean velocities, when flowing full, of not less than 0.6 m/s (2.0 ft/s). The following minimum slopes should be provided.

Sewer Size (mm)	Sewer Size (inches)	Minimum Slope (%)
100	4	2.0
150	6	0.6
200	8	0.40
250	10	0.28
300	12	0.22
380	15	0.15
460	18	0.12
530	21	0.10
610	24	0.08
690	27	0.07
760	30	0.06
920	36	0.05

Minimum Slope By Sewer Diameter
Figure 9-4.1

When a gravity line is not possible, the second best choice is connecting into a municipal system via a packaged sewage lift station and a short pressure line. Metering of a lift station can usually be accomplished by placing a counter on the pumps and calibrating the volume of sewage discharged by each cycle of the pump.

9-4.2 Long Distance Pressure Sewers

When a pressure sewer line is longer than a few thousand feet, the designer needs to be aware of special problems that can occur. Long runs of pipe can significantly increase the detention time of the effluent in the pipe, which can cause the effluent to turn septic. This septic sewage will generate hydrogen sulfide which is noted for its toxicity and for its ability to cause corrosion of many materials used in sewer construction.

The hydrogen sulfide gas has also been known to cause an odor nuisance at the point where it is released to the atmosphere.

Long pressure sewers must be constructed of PVC plastic pipe in order to resist the corrosive effects of hydrogen sulfide gas. The ventilation points should be remotely located to avoid becoming an odor nuisance.

The designer should strive to discharge into a sewer with a fairly significant base flow. This will allow the septic sewage to dilute with fresh sewage and thereby cause less damage to the receiving sewer system. When connecting to a local sewer system, two to three manholes downstream of the connection should be covered with hydrogen sulfide resistant coating. Discharging into the remote end of a concrete sewer should be avoided.

Mechanical aeration should be considered as a treatment to the hydrogen sulfide problem. Several references, such as the Department of Ecology's *Criteria for Sewage Works Design,* are available to aid the designer in sizing aeration equipment for the pressure line as well as the wet wells. Other methods of treatment, such as chemical additives, are not recommended due to the costs and operational problems.

Sediment build-up is also a great concern with long distance pressure sewers. For this reason, it is recommended that the sewage be passed through a septic tank prior to entering the pressure line. This will greatly increase the reliability of the pumps and will also minimize the sedimentation problems. The fact that the sewage is septic is not a major concern since it will turn septic anyway when placed in a pressure main that has over a 24-hour detention period. The designer should keep in mind that detention times will be even longer with normal flow rates than they would be with the peak design days.

The designer should consider the effects of "air-binding" or "air-locking" in a pressure line especially when the line has an excessively undulating profile. Air release valves are effective in handling air pockets and should be installed in a section of pipe that slopes up toward the hydraulic grade line or runs parallel to it. For long parallel runs, the designer should consider installing the air release valves every 450 meters (1,500 feet).

For maintenance purposes, sewer cleanouts should be installed at a maximum spacing of 100 meters (300 feet). When considering a long distance pressure sewer, it is recommended that the designer work closely with the OSC Hydraulics Branch from the earliest stages of design.

9-4.3 Septic Tank and Drainfield

In addition to the items listed in Section 9-2.2, the design report for a septic tank and drainfield shall also include the following items:

1. **Sieve Analysis and Hydrometer Test:** Tests to be performed on soil samples taken in the immediate area of and at the depth of the proposed drainfield. Results to be used to determine soil type.
2. **Very Descriptive Soil Profile:** The profile description must include at least 1.2 m (4 feet) of the soil strata below the bottom of the proposed trench.
3. **Area Drainage:** Drainfield shall be located in such a manner as to prevent interference with surface drainage and contamination of subsurface drainage.

   (See Section 9-4.3.2 for setback requirements.)
4. **Water Table During "Seasonal Wet Periods:"** The water table during "seasonal wet periods" shall not be higher than 1.2 m (4 feet) below the bottom of the trench.
5. **Special Provisions:** They shall be written in such a manner as to limit the equipment size for work within the drainfield area so as to not unduly disturb the existing soil characteristics.

The Environmental Protection Agency (EPA) *Design Manual — On-site Wastewater* Treatment and Disposal Systems outlines the basic principles which should be followed for the design, construction, and maintenance of a septic tank and drainfield. In addition, the designer should work closely with the appropriate governmental regulatory agency which will review and has the authority to approve the design.

The appropriate health authority would be one of the following:

Design Flow (m^3/d)* Metric	Design Flow (gpd)* English	Review Agency
0 - 13.25	0 - 3,500	Local County Health Department
13.25 - 54.9	3,500 - 14,500	Washington State Dept. of Health or Local Health Department, if approved by the Washington State Dept. of Health to represent them.
Over 54.9	Over 14,500	Department of Ecology

*"Design Flow" is based on highest "future peak day usage."

9-4.3.1 Septic Tank Sizing

The size of the septic tank shall be determined from the projected peak daily use. The septic tank shall have a volume equal to 1.5 times the volume of sewage generated during the peak design day. This will allow storage volume for solids as well as provide a minimum detention time of 24 hours.

Where RV dumps are present, care should be taken to keep the RV dump sewage separate from the rest room sewage. There is an uncertainty of what is actually deposited at these dump sites since these sites are not monitored. Because RV owners use several chemical preservatives for odor control and disinfection, these preservatives are known to be present in RV dump sewage. These preservatives cause an imbalance in the organic breakdown of the sewage resulting in high waste strengths. Due to the uncertainty and the high waste strengths, separate septic tanks for the RV dump waste should be used.

The septic tank for RV sewage shall have a volume equal to three times the volume of sewage generated during the peak design day. This will allow storage volume for solids as well as provide a minimum detention time of 72 hours. Shorter detention times may be used if approved by the appropriate governmental regulatory agency.

Septic tanks shall be constructed with two compartments. The first shall consist of two-thirds of the total required volume and the secondary compartment shall be one-third. When upgrading an existing rest area, the designer may use more than two compartments and the to ratio may be modified, however, the largest compartment should be placed first when possible.

Standard Plan I-1 shall be used for the construction of all septic tanks at highway rest areas. If precast or fiberglass septic tanks are used, the designer should make sure the proposed tanks are approved for use by the local or state health department. The designer should include an effluent screen at the outlet of the septic tank to prevent any large particles from being transported to the drainfield.

9-4.3.2 Drainfields

A drainfield consists of a distribution pipe and gravel installed in original undisturbed soil for the purpose of transmitting effluent into the soil.

The drainfield must be placed in a suitable soil which has a minimum depth of 1.2 m (4 feet) below the bottom of trench. There should be a minimum of 1.2 m (4 feet) between the bottom of the trench and the water table. If minimum vertical separation can not be met, above ground pretreatment devices such as sand filters may be required prior to discharging effluent to the drainfield. The designer should consult with the appropriate health authority to determine what is acceptable.

Soils should be classified by normal laboratory and field procedures according to the following figure:

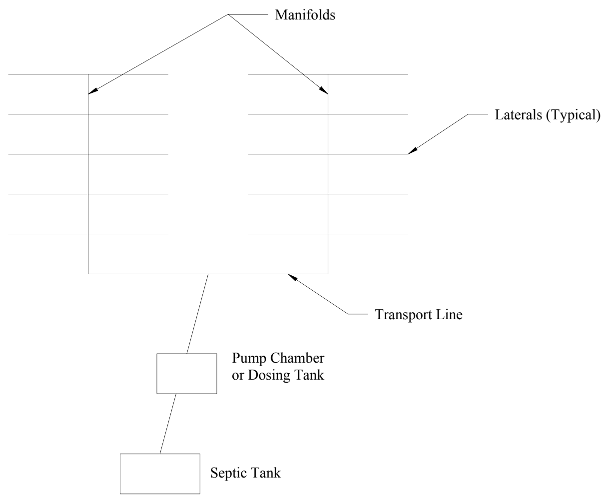
USDA Soil Type	Application Rate ($m^3/day/m^2$) (metric)	Application Rate ($gal/day/ft^2$) (English)	Approximate Percolation Rate (min./mm) (metric)	Approximate Percolation Rate (min./in) (English)	Soil Textural Classification
1A & 1B	Unsuitable	Unsuitable			Very gravelly coarse to very fine sands, very gravelly loamy sands
2A	0.049	1.2	0.039	1	Coarse sands or gravels
2B	0.049	1.2	0.039 - 0.157	1 - 4	Medium sand
3	0.033	0.80	0.197 - 0.354	5- 9	Fine sand, loamy coarse & medium sand
4 0.	0.024	0.60	0.394 - 0.748	10 - 19	Very fine sands, loamy fine & very fine sands, Sandy loam, Loam
5	0.018	0.45	0.787 - 1.14	20 - 29	Porous, well developed structure in silt and silt loams
6	Unsuitable	Unsuitable	1.18	30	Other silt loams, silty clay loams, and clay loams

Soil Texture Classifications
Figure 9-4.3.2A

Soil Types 1A, 1B, and 6 are classified as unsuitable soils. Soil Types 1A and 1B are very gravelly and have very high percolation rates. These soil types are considered unsuitable because it is possible that the effluent will infiltrate through the soil at a rate that will not provide proper treatment. Drainfields can be constructed in areas with these types of soils if precautions are taken to provide proper treatment of the effluent. The designer should consult with the appropriate health authority to determine what is acceptable. Type 6 soils are considered unsuitable because of the very low percolation rates. Effluent is unable to effectively infiltrate through this type of soil.

The area of drainfield required to adequately dispose of the sewage generated from the rest area is calculated by dividing the volume of sewage generated daily by the application rate. Drainfields usually consist of 0.9 m (3 feet) wide trenches on 2.3 m (7.5 feet) centers. However, in soils with a texture of Type 1A, 1B, 2A, or 2B, absorption beds, which are trenches that are greater than 0.9 m (3 feet) wide, are allowed. The maximum width of an absorption bed is 3 m (10 feet). The minimum spacing between beds is 6.1 m (20 feet).

The plumbing details for a system like this are very site specific and must be worked out with the responsible health agency. The general layout would include gravity flow from septic tanks to dosing tank or pump chamber. Pressure flow from the dosing tank or pump chamber is carried to the drainfield by a transport line from which a manifold conveys the flow to laterals. The effluent is discharged into the ground through orifices in each lateral.



General Drainfield With Septic Tank Layout
Figure 9-4.3.2B

If 0.9 m (3 feet) wide trenches are used, the designer should attempt to minimize excavation. This can be accomplished by designing the trenches parallel to the contours of the land where the drainfield is to be built. These trenches can also be designed in a terraced fashion with each trench or group of trenches designed at a particular elevation.

The discharge rates through each orifice and the friction losses through each pipe segment must be calculated to ensure even distribution of the effluent throughout the drainfield. EPA requirements include: orifice discharge rates from one end of the lateral to the other can not vary by more than 10 percent and total head loss due to friction in the manifold can not exceed 10 percentof the head loss at the far end of the manifold (the farthest point from the transport line). Thus, the orifice discharge controls the maximum lateral length, while the head loss between laterals limits the maximum manifold length. See the EPA *Design Manual* for criteria in calculating orifice discharges and head losses.

Minimum setback distances must be maintained from natural and man-made features. The following figure can be used as a guide for determining the general location of a drainfield. Because the setback distances could vary from area to area, the designer should check with the governing health authority.

When the amount of trench exceeds 150 m (500 feet) a dosing tank with a siphon should be used in conjunction with the septic tank. This will allow for proper distribution of the sewage throughout the drainfield and also allows the drainfield to rest or dry out between doses. Allowable dosing frequencies are dependent on soil type and provided in the EPA

Design Manual. When the amount of trench exceeds 300 m (1,000 feet), a dosing tank with alternating siphons discharging into separate drainfields should be used. A pump should also be considered for these larger systems when the hydraulic gradient is insufficient for a dosing siphon.

Item	From Edge of Drainfield		From Septic Tank and D-Box		From Building Sewer	
	(m)	(ft)	(m)	(ft)	(m)	(ft)
Well	30	100	15	50	15	50
Well Water Supply Line	3	10	3	10	3	10
Surface Water	30	100	15	50	3	10
Building	3	10	1.5	5	0.5	2
Right of Way Line	1.5	5	1.5	5	1.5	5
Drainage Ditch (Upslope)	3	10	N/A	N/A	N/A	N/A
Drainage Ditch (Downslope)	10	30	1.5	5	N/A	N/A
Cuts or Banks — 5' min. suitable soil depth	8	25	N/A	N/A	N/A	N/A
less than 5' soil depth	15	30	N/A	N/A	N/A	N/A

Minimum Horizontal Setbacks
Figure 9-4.3.2C

Septic Tank and Drainfield Example:

The following example will serve only as a guide to those involved in the design of a septic tank and drainfield. Many special problems may occur during the design which must be solved by the engineer's own judgment or by the judgment of the appropriate health officials.

Septic Tank Design:

$$\text{Volume From Rest Rooms} = 1{,}850\,\frac{\text{persons}}{\text{day}} \times 0.013\,\frac{\text{m}^3}{\text{persons}} = 24\,\frac{\text{m}^3}{\text{day}}$$

Required Septic Tank Volume = 24 x 1.5 = 36 m^3

Use one 38 m^3 (10,000 gallon) septic tank for restroom wastewater.

$$\text{Volume From RV Dump} = 50\,\frac{\text{RVs}}{\text{day}} \times 0.3\,\frac{\text{m}^3}{\text{RV}} = 15\,\frac{\text{m}^3}{\text{day}}$$

Required Septic Tank Volume = 15 x 3.0 = 45 m^3

Use one 45 m^3 (12,000 gallon) septic tank for RV dump wastewater.

Drainfield Design — The engineer and the Health Department have agreed that the native soil is a type 3 soil and has a suitable depth of at least 1.2 m (4 feet) below the bottom of the drainfield. The application rate for a type 3 soil is 0.033 m^3/day/m^2 (0.80 gpd/ft^2).

The size of drainfield required is:

$$\frac{24+15\ m^3/day}{0.033\ m^3/day/m^2} = 1{,}182\ m^2\ (12{,}723\ ft^2)$$

If a 0.6-m (2-foot) wide trench is used, this would result in 1970 m (6,360 LF) of drained trench.

If a 0.9-m (3-foot) wide trench is used, this would result in 1314 m (4,241 LF).

If an absorption bed is used, the designer should provide 1182 m^2 (12,723 ft^2) of bottom area. This would be 10 rectangular beds of 40-m long by 3-m wide (131-feet long by 10-feet wide), or any other combination of rectangular shape having the same area and not exceeding the 10-feet maximum width. Six meter (20-feet) spacing between beds is required.

The following is a list of details which must be addressed in the hydraulic report:

Gravity vs. siphon vs. pump
Dose volume
Volume inside piping system
Minimum orifice size and spacing
Pipe diameters and length
Number of doses per day
Achieving equal distribution
Manifold sizing
Transport line sizing
Maximum and minimum cover
Setback distances

The EPA manual referenced in Section 9-4.3 along with Washington State Department of Health *Design Standards for Large On-Site Sewage Systems* and *Guidelines for the Use of Pressure Distribution System* are essential when deciding on these details.

9-4.3.3 Other On-site Disposal Systems

Septic tank and drainfield has been the traditional manner in which to treat wastewater when public sewers were not available. Unfortunately, there are soil and site conditions that are unsuitable for these conventional systems such as poor infiltrative soils, or close proximity to groundwater. As a result, alternative systems are used in conjunction with or in lieu of the conventional systems. Examples of alternative systems include sand filters and mounds. The designer should refer to DOH's *Guidelines for Sand Filters, May 1995 and Guidelines for Mound Systems, September 1993* for detailed information regarding the use and design of these alternative systems.

The designer should work very closely with the regulating health authority to ensure that the system chosen is an acceptable and approved alternative.

9-4.4 Sewage Lagoons

Due to the excellent climatic conditions in Eastern Washington, the use of sewage lagoons is considered to be the best method of sewage disposal. The high evaporation rates and low precipitation rates are very conducive to the successful operation of a lagoon.

Normally, a nondischarging sewage lagoon is designed for its BOD loading (Biochemical Oxygen Demand). The designer must size the lagoon such that there is a balance between BOD loading, nitrogen loading, and hydraulic loading. The hydraulic loading is the rest area effluent and precipitation. The Department of Ecology's *Criteria for Sewage Works Design* should be consulted when designing a sewage lagoon.

Even though RV sewage is known to have high waste strengths, the BOD strength of the effluent from WSDOT rest areas is usually low since rest room usage exceeds RV Dump usage. Under these circumstances, the lagoons for a rest area can be designed on an inflow-outflow principle with a final check on BOD loading.

Basically, the amount of precipitation plus the amount of effluent must be equal to the amount of evaporation. The amount of effluent is based on the number of persons using the rest area on an average day in each month. The precipitation and evaporation rates must be determined from climatological records which have been assembled by various U.S. Weather Bureau stations. This information is available at the OSC Hydraulics Branch. The BOD loading is then checked.

Lagoons that hold more than 12,335 m^3 (10 acre-ft) of effluent are considered dams. The designer must complete the dam safety analysis procedure dictated by DOE. The time required to complete the analysis and obtain DOE approval is four months. OSC Hydraulics Branch should be contacted to assist in the design.

An example problem is included which gives the allowable loading rates and the procedures to be followed in the design of a lagoon. The allowable loading rates developed from various studies range from 1.68 to 5.59 grams/m^2/day (15 to 50 lbs/acre/day). Based on WSDOT's use of sewage lagoons at rest areas, the designer should use a pond loading of 4.47 grams/m^2/day (40 lbs/acre/day).

9-4.4.1 Design Example

Design Year — 2017
Flow — 0.013 m^3/person (3.5 gallons/person)
BOD Loading — 3.17 grams/person/day (0.007 lbs/person/day)
Pond Loading — 4.47 grams/m^2/day (40 lbs/acre/day)

Month	Persons Using Rest Rooms on Average Day by Month	RV's Using RV Dump on Average Day by Month
January	450	8
February	600	11
March	1,000	18
April	1,550	38
May	1,700	51
June	2,250	68
July	2,350	85
August	2,450	89
September	1,900	69
October	1,350	41
November	1,050	19
December	600	11

Climatological Data:

Month	Precipitation (mm.)	Evaporation (mm.)
January	23.11	Negligible
February	26.16	Negligible
March	23.62	Negligible
April	21.34	150.62
May	14.99	202.95
June	8.89	236.98
July	4.83	304.55
August	3.56	255.78
September	10.92	159.26
October	18.80	75.18
November	24.38	Negligible
December	24.13	Negligible

Effluent from Rest rooms and RV Dump:

$$Qs = [30 \frac{\text{days}}{\text{mo.}} \times 0.013 \times \frac{m^3}{\text{person}} \ N] + [0.3 \ \frac{m^3}{RV} \times RV\ ] = 0.4\ N + 0.3\ RV$$

where N = persons using rest rooms daily and RV = RV's using RV dump on an average day.

Precipitation and Evaporation:

$$Q = 10{,}000 \frac{m^3}{\text{hectare}} \times \frac{m}{1000\ m} \times R\ (\text{mm})$$

$$= 10\,R \frac{m^3}{\text{hectare}} \quad \text{where } R = \text{precipitation and evaporation rate.}$$

Lagoon Calculations:

Month	# of Persons Using the Rest Rooms	# of RV's Using the Rest Area	Effluent (m^3/Mo.) Qs	Precip. Rate (mm)	Precip. (m^3/hectare) QP	Evap. Rate (mm)	Evap. (m^3/hectare) QE
JAN.	450	8	182.4	23.11	231.1	Neg.	0
FEB.	600	11	243.3	26.16	261.6	Neg.	0
MAR.	1,000	18	405.4	23.62	236.2	Neg.	0
APR.	1,550	38	631.4	21.34	213.4	150.62	1506.2
MAY	1,700	51	695.3	14.99	149.9	202.95	2029.5
JUNE	2,250	68	920.4	8.89	88.9	236.98	2369.8
JULY	2,350	85	965.5	4.83	48.3	304.55	3045.5
AUG.	2,450	89	1006.7	3.56	35.6	255.78	2557.8
SEPT.	1,900	69	780.7	10.92	109.2	159.26	1592.6
OCT.	1,350	41	552.3	18.8	188.0	75.18	751.8
NOV.	1,050	19	425.7	24.38	243.8	Neg.	0
DEC.	600	11	243.3	24.13	241.3	Neg.	0
TOTAL			7052.4		2,047.3		13,853.2

Note: Precipitation and evaporation is measured in volume per unit area. The effluent from the rest rooms and the RV dump is a direct measurement of volume. Therefore, precipitation and evaporation measurements must be multiplied by an area to be comparable to the rest room and RV dump volumes.

Inflow-Outflow Principle: Inflow = Outflow (1)

Inflow = Rest Room Volume + RV Dump Volume + Precipitation x Lagoon size (area) (2)

Outflow = Evaporation x Lagoon size (area) (3)

Substituting equations (2) and (3) into equation (1),

Rest Room Volume + RV Dump Volume = Lagoon size x (Evaporation – Precipitation)

$$\text{Lagoon size} = \frac{\text{Rest Room Volume} + \text{RV Dump Volume}}{\text{Evaporation} - \text{Precipitation}} \quad (4)$$

Substituting the values calculated in the figure into equation (4):

$$\text{Lagoon size} = \frac{7{,}052.4}{13{,}853.2 - 2{,}047.3} = 0.6 \text{ hectares (1.48 acres)}$$

Use a 0.6 hectare (1.5 acres) lagoon.

Check BOD Loading:

Check for peak day usage of rest area

Restroom: ADT = $9{,}000 \frac{\text{cars}}{\text{day}}$ 0.198 (% cars entering rest area on a peak day) x 2.36 $\frac{\text{persons}}{\text{car}}$ 0.7 (% persons using the restroom) = <u>2,950 persons/day</u>

BOD = 3.17 grams/person/day x 2,950 $\frac{\text{persons}}{\text{day}}$ = <u>9,351.5 grams/day</u>

Area required = 9,351.5 $\frac{\text{grams}}{\text{day}}$/4.47 grams/m^2/day = <u>2,092 m^2</u>

RV Dump: ADT = 9,000 $\frac{\text{cars}}{\text{day}}$ x 0.198 (% cars entering rest area on a peak day) x 0.05 (% cars entering rest area that are RV's) x 0.303 $\frac{\text{m}^3}{\text{RV}}$ / 0.013 $\frac{\text{m}^3}{\text{person}}$ = 2,077 persons/day

BOD = 3.17 grams/person/day x 2,077 $\frac{\text{persons}}{\text{day}}$ = <u>6,584 grams/day</u>

Area required = 6,584 $\frac{\text{grams}}{\text{day}}$/grams/m^2/day = <u>1,473 m^2</u>

Total acreage needed for BOD treatment = = 2,092 + 1,473 = 3,565 m^2
= 0.36 hectare < 0.6 hectare
OK

9-4.4.2 Construction of Sewage Lagoons

It is recommended that a cellular type of lagoon, with a minimum of two cells and with provisions for future expansion be constructed in rest areas. In most lagoon designs, the size of the lagoon is based on the 20-year volumes. The designer should design the first and maybe second cells (if more than two cells are designed) to meet current usage. This type of lagoon allows for a better regulation in the operating depth of sewage, meets future needs, and ensures that sufficient volumes are available for proper treatment.

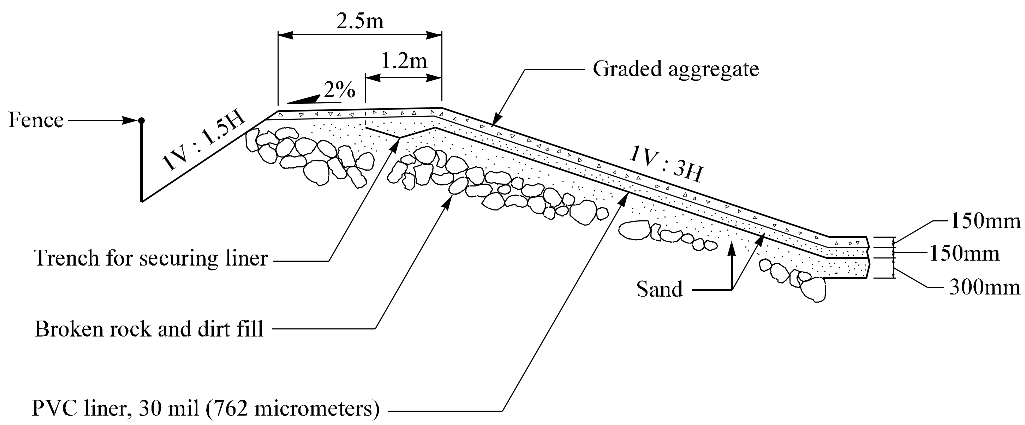
An overflow weir should be constructed in the dike separating the cells, at a water depth of 1.5 m (5 feet) which is the maximum allowable operating depth. In order to control the operating levels of the cells, a 150-mm (6-inch) pipe with a gate valve should be installed at a water depth of 0.9 m (3 feet). Once the depth of sewage reaches 0.9 m, the sewage will flow through the pipe to the next cell. This gate valve will only allow one way flow from the first cell to the second cell through the 150-mm pipe. If the second cell exceeds the 0.9-m depth, the gate valve will block the flow from going back to the first cell.

The inlet pipe from the rest area should discharge at the bottom of the lagoon, preferably near the middle of the cell. A concrete apron or splash block should be constructed to minimize erosion at the point of discharge.

The embankments or dikes should be constructed of a relatively impervious material and be sealed with a PVC lining, with welded joints, to prevent seepage of the sewage into the ground. The lining should be laid on a 300-mm (12-inch) sand blanket and then covered with another 150 mm (6-inch) sand blanket as shown in Figure 9-4.4.2. It is recommended that the embankments have a minimum top width of 2.5 m (8 feet) to permit access for maintenance vehicles.

The inner slopes should be constructed on a slope 1(V) to 3(H) and be provided with a freeboard depth of 0.9 m (3 feet). The slopes should be protected with a fractured rock to guard against possible erosion due to wave action.

The sewage lagoon should be protected from the public by the use of a 1.5-m or 1.8-m (5-foot or 6-foot) high fence, preferably the chain link type.



Typical Embankment Section
Figure 9-4.4.2

9-4.4.3 Maintenance of Sewage Lagoons

The maintenance on a sewage lagoon is very minimal, however, it is very important in order to ensure continued operation of the system. The principal consideration of maintenance is ensuring the operating depths are within the recommended operating range. All vegetation must be controlled around the lagoon because the roots of many plants will damage the liner. The control of weeds is also necessary so that breeding places for mosquitoes can be eliminated. The embankments should be checked periodically and any repairs due to erosion or burrowing rodents should be made.

The control of odors should not be a problem except possibly a small amount of odor may be detected in the spring after the lagoon has thawed. If it is too objectionable, sodium nitrate can be added to reduce the odors.